Proceedings of the Nineteenth Dredging Seminar

Ocean Engineering

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Dredging Applications of High Density Polyethylene Pipe

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Introduction

Since its development in the early 1950s, high density polyethylene (HDPE) pipe has gained wide acceptance as the material of choice in many applications. A combination of physical and chemical properties make HDPE pipe resistant to corrosion, abrasion, deformation, and water hammer effects. It is also lightweight and flexible and has excellent flow characteristics.

One area in which HDPE pipe use is increasing is slurry discharge lines. Laboratory tests and actual field use have shown that HDPE pipe will outlast steel pipe for slurry transport under certain conditions. At the U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Miss., a dredging test loop was constructed in the Hydraulics Laboratory. Two conditions of slurry velocity and concentrations (homogeneous and heterogeneous) were run with steel and HDPE pipe and elbows. The test results confirmed industry claims that depending on slurry flow and concentration, HDPE pipe is three to five times more abrasion-resistant than conventional mild steel.

Although the test results were encouraging, field data were needed to determine the effectiveness of HDPE pipe in a full-scale dredging operation. With the cooperation of the Operations Division, U.S. Army Engineer District, Portland, and the Port of Portland, two 20-foot-long sections of 30-inch-inside diameter (ID) HDPE pipe were installed in the land discharge line of the dredge OREGON. The pipe was installed on August 20, 1985, and was in use until February 26, 1986. Wall thickness measurements were taken to measure wear by the amount of wall thinning. After the pipe handled over 2.6 million cubic yards of dredged Columbia River material, wall thickness measurements taken at various points along the pipe length showed wear from 1.1 percent to 29.9 percent of the original thickness. Wear patterns were more closely associated with the material type, i.e., coarse sands and gravel, than with the amount of material dredged.

This paper discusses some of the properties of HDPE pipe that make it especially applicable to dredging situations, the results of the Portland HDPE pipe field test, and some comments and views on HDPE pipe by dredge operators who use the product. The information in the following two sections, "High Density Polyethylene Pipe" and "Dredging Applications," was extracted from the manufacturers' literature listed in the bibliography.

High Density Polyethylene Pipe

High density polyethylene plastic is made from an extremely tough and durable resin with a very high molecular weight. These polyethylenes are formed by the polymerization of a group of straight chain, unsaturated hydrocarbons (ethylenes) into long-chain molecules. In the late 1970s an ultra-high molecular weight high density polyethylene product was developed. Careful programming of the polymerization reaction can control the density and molecular weight to obtain HDPE pipe compounds with desired characteristics. American Society for Testing and Materials (ASTM) Standard D 1248, "Polyethylene Plastic Molding and Extrusion Materials," although revised several times to meet the needs of industry, was not adequate for all HDPE materials. As a result, ASTM D 3350, "Polyethylene Plastic Pipe and Fittings Materials," was written especially for very high molecular weight materials. This new system uses cell classification rather than a material code to designate the polyethylene property. Using the cell classification system of ASTM D 3350, a material classified as PE355434C (available as Driscopipe 8600), has the following properties (the code "PE" indicates polyethylene:

Property	Cell Classification
1. Density, g/ccm (base resin)	0.941 - 0.955 (cell 3)
2. Melt Index	Flow rate, 4.0 g/10 min. by condition F. Method D 1238, (cell 5)
3. Flexural Modulus, psi	120,000 to 160,000 (cell 5)
4. Tensile Strength @ Yield, psi	3000 - 3500 (cell 4)

5. Environmental Stress Crack Resistance — Test Condition C, 100°C, 192 hr (cell 3)
6. Hydrostatic Design Basis @ 23°C (73.4°F), psi — 1600 (cell 4)
7. Color and Ultraviolet Stabilizer Code — C (Black with 2 percent minimum carbon black)

Other manufacturers have comparable products with ASTM D3350 classifications. Nipak "Custom HD" PE3408 has a cell classification of PE345534C, while the Plexco product PE3408 is classified as PE345434C.

Dredging Applications

Certain properties of HDPE pipe make it applicable to dredging situations. Use of this pipe can provide cost savings in installation labor and equipment, maintenance, freedom of design, and extended life of pipeline systems. HDPE pipe has a high potential for application in the marine environment because it will not rot, rust, or corrode; conduct electricity; nor support growth of or be affected by algae, bacteria, or fungi; and is resistant to marine biological attachment. Its specific gravity of 0.955-0.957 makes HDPE 70 to 90 percent lighter than concrete, cast iron, or steel pipe, requiring greatly reduced manpower and equipment for transportation and installation. The extremely smooth inside surface and non-wetting characteristic of HDPE result in higher flow capacity and reduced friction loss. For example, a pipe coefficient or "C" factor of 155 is used in the Hazen-Williams formula for fluid flow calculations for HDPE pipe sections. The "C" factor for new steel is 140 and for old steel, 125. Since the "C" factor value is inversely proportional to head loss due to friction, a high "C" value is desirable. Although HDPE pipe can be joined by flanges or compression couplings, the heat fusion technique–butt fusion–is recommended. This process produces a joint of high integrity and reliability that is as strong as the pipe in both tension and hydrostatic loading. HDPE pipe cannot be joined by solvent cements or adhesives.

Another HDPE property that makes it suited to dredging applications is its abrasion resistance. Controlled laboratory tests have shown that HDPE pipe outperforms steel pipe by a ratio of 4 to 1. It can be stored outside for years without danger of damage by ultraviolet exposure because of the carbon black content of the material. The flexible nature of HDPE enables it to absorb impact loads, surge pressures, vibrations and stresses. It can be cold bent in the field to a minimum radius of 20 to 40 times the pipe diameter, and can easily conform to uneven ground contours.

All thermoplastic piping materials are affected by changes in temperature. Normal temperature changes do not cause degradation but may affect the physical and chemical properties of the material. It is the general industry practice to characterize HDPE material at ambient temperature, 73.4°F (23°C). As temperature increases, long-term strength decreases and vice versa. For example, at 73°F (23°C) HDPE long-term strength is 1,600 pounds per square inch (psi); at 120°F (49°C) it is 1,000 psi; and at 50°F (10°C) it is 1,824 psi. The material will soften at 260°F (127°C) and become molten at 475°F (246°C). Although the temperatures of dredging slurries are usually not extreme, the air temperature and the amount of radiant sunlight heat might be a consideration in some environments.

Field Test Preparations

The purpose of this field test was to determine the feasibility of using large-diameter HDPE pipe in a dredging application. Smaller diameter HDPE pipe, up to about 12 inches in diameter, has been successfully used in dredging for several years, and its use is becoming more widespread. Preliminary conversations with HDPE manufacturers' engineers indicated that the proposed use of large-diameter HDPE pipe in the discharge line of a dredge was a valid one. For ease and economy of shipping and handling, two 20-foot sections of 30-inch ID (32-inch nominal size outside diameter (OD)), Standard Dimension Ratio (SDR) 32.5, Driscopipe 8600 with stub ends and steel back-up rings on each end were purchased. These were factory fabricated and delivered to the Portland site in early May 1985.

Installation

The HDPE pipe sections were installed in the discharge line of the Port of Portland dredge OREGON. The OREGON is a cutter-suction type, non-propelled, 30-inch hydraulic pipeline dredge, with steel hull and superstructure. It was built in 1965 by Bauer Dredging Company, and rebuilt in 1979 by Northwest Marine Iron Works, Inc. The OREGON performs dredging in the Lower Columbia and Lower Willamette rivers and is capable of handling 2,000 to 3,000 cubic yards of dredged material per hour.

Unusually low sediment deposition delayed the dredging effort by several months. Dredging was finally started in August 1985, and the HDPE pipe sections were marked, measured and installed. The method used to determine pipe wear was to measure wall thickness at discrete locations along and around the pipe with a lightweight, portable ultrasonic thickness gauge. The outside of the pipe sections was coded and marked every 2 feet along the length and at eight equally spaced locations around the circumference, for a total of 160 locations on the two pipe sections at which wall thickness measurements were taken (Figure 1).

The outer two ends of the HDPE pipe were fitted with quick-connect couplings. Steel flanged sections, about 3 feet long, were fabricated and bolted at one flanged end of each HDPE section. One of the fabricated steel sections was slightly tapered so it could fit inside the adjoining straight section of steel pipe for rapid pipeline deployment (Figure 2). The Port of Portland uses the quick-connect method on all land discharge pipe. Initially, the inner flanges of the HDPE pipe were bolted together. However, it was time-consuming to align the HDPE to HDPE flanges properly, and, in time, these ends were also fitted with the quick-connect fittings.

Prior to testing, wall thickness measurements were taken to establish a baseline for wear rates. All external scrapes and gouges were noted and photographed, and a thorough inspection of the interior surface was made. It was recommended that the HDPE sections be given no special treatment beyond that necessary to utilize the pipe, but that care be taken, when using heavy equipment, not to severely gouge or damage the pipe. The placement of the HDPE sections in the discharge line was such that they were always on the land, rather than in the floating pipeline sections. They were also placed at a distance from the dredge pump so the pipeline pressure at the HDPE sections would be less than the rated 50 psi.

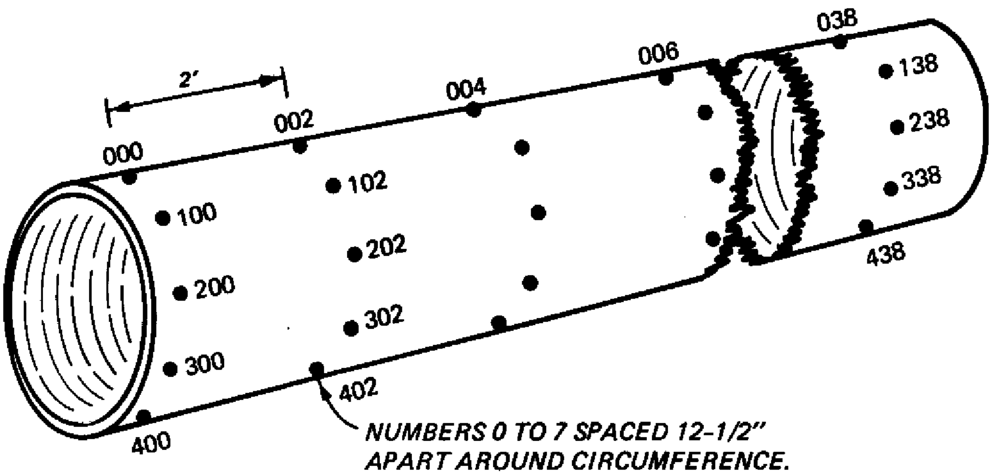


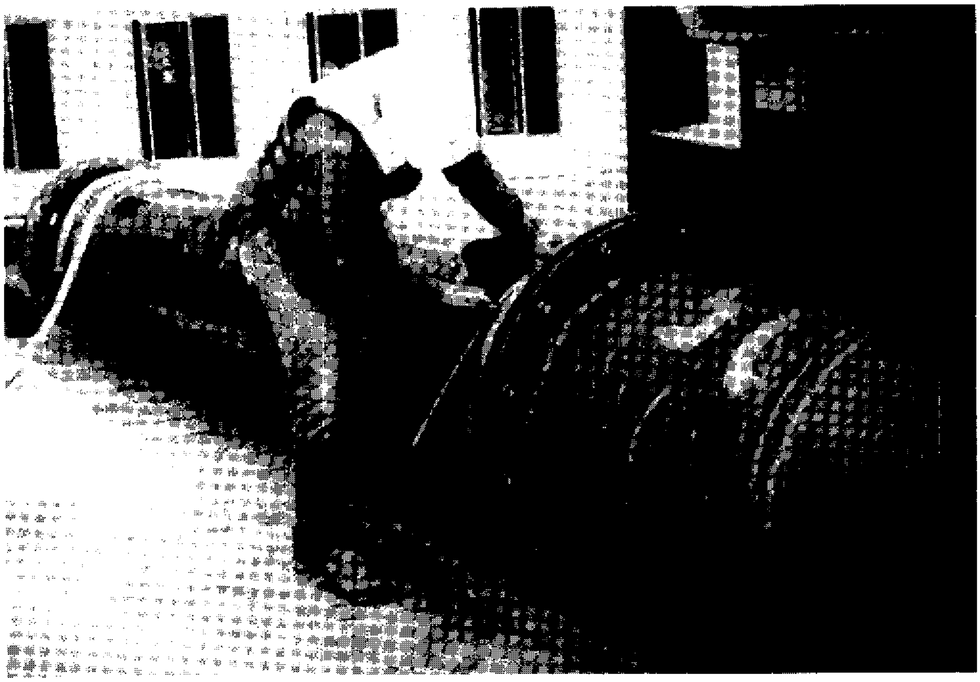
Figure 1. Wall thickness measurement numbering scheme.

Measurements

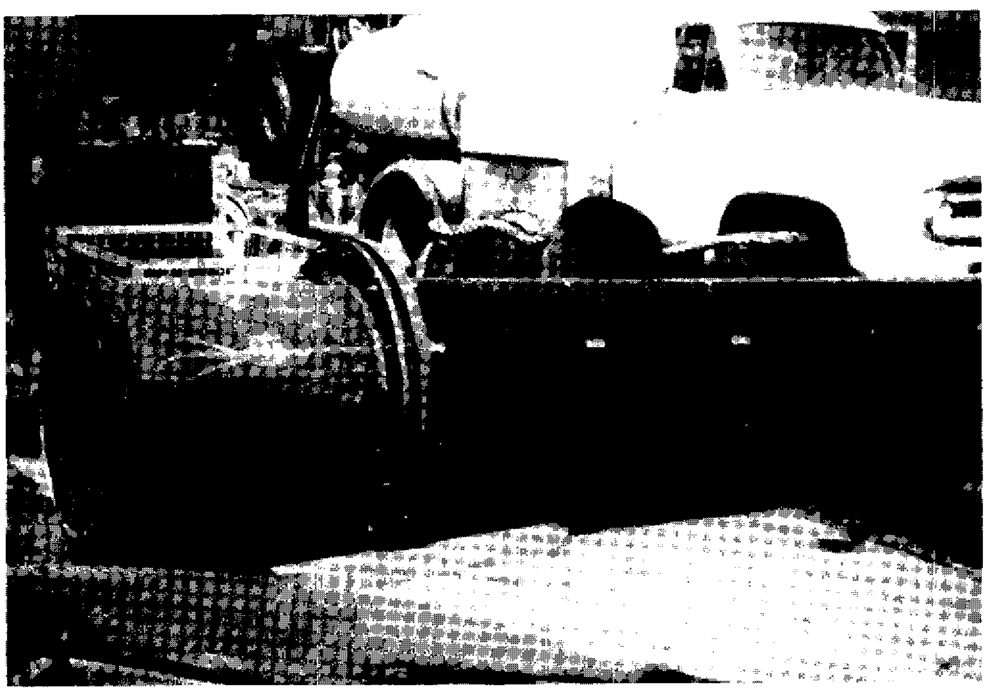
When the HDPE pipe sections were in the discharge line, wall thickness measurements were taken when the dredge was not operating to avoid any meter reading irregularities that might be caused by the moving slurry. In addition to wall thickness, information was recorded on (a) the row number appearing on the top of the pipe; (b) the distance of the HDPE sections from the pump; (c) pumping pressure; (d) slurry velocity, concentration, and type (i.e. sand, gravel); and (e) dates and time the test sections were on-line.

Throughout the dredging season the HPDE sections were always next to each other, and aligned so the number that appeared on top in Section 1 also appeared on top in Section 2. Flow was always in the same direction and entered the HDPE sections through Section 1. All wall thickness measurements were taken by the same person, and the number on top was recorded to document the rotation of the pipe.

Average slurry concentration during the test 15 percent, and the material ranged from fine sands to 2-in-diameter smooth river rock, with the majority being coarse sand. Slurry velocity averaged 18 feet per second (fps) with a range from 13 to 25 fps. Eleven sets of wall thickness readings were taken during the August 1985 to February 1986 dredging period. The HPDE sections were on-line for 126 days as listed in Table 1.



a. Tapered end



b. Straight section

Figure 2. Quick-couplings bolted onto outer ends of HDPE sections

Table 1. HDPE Pipe Data - Days On-Line and Amount Dredged

Pipe On-Line	Data Taken	Pipe Out	Days On-Line	Bottom No.	Amount Dredged Cu. Yards
1985					
8/20	8/20 BASE	8/22	3	6	396,880
8/26	8/26, 9/1	9/17	23	5	399,316
9/25	10/2	10/17	23	3	436,612
10/25	10/30	10/30	6	4	115,783
11/4		11/8	5	2	57,285
11/8	11/10, 16, 23	12/6	28	2	330,437
12/10	12/7	12/16	7	1	181,025
12/17		12/19	3	0	136,534
1986					
1/9	1/8	1/21	13	7	138,266
1/22		1/28	7	6	289,878
2/18	3/4 Final	2/25	8	7	177,725
Total			126		2,659,741

The assumption was made that pipe wear would be greatest on the bottom. Table 2 lists the amount of material that was dredged when each of the eight locations was located on the bottom.

New HDPE pipe of this size has a manufacturer's guaranteed wall thickness of 0.969 inch. The baseline data revealed that all readings were greater than 0.969 inch and averaged 1.018 inch. The accuracy of the wall thickness gage is ±0.005 inch.

Table 2. Location on Bottom of HDPE Pipe and Amount Dredge

Bottom Number	Cubic Yards Dredged	Percent of Total
0	136,534	5.13
1	181,025	6.81
2	387,722	14.58
3	436,612	16.42
4	115,783	4.35
5	399,316	15.01
6	686,758	25.82
7	315,991	11.88
Total	2,659,741	100.00

Results

The almost 1,760 wall thickness values were examined closely for trends in pipe wear. Flanged pipe may experience greater wear due to the turbulence created by the flange at a distance equal to 4 to 10 pipe diameters from the flange. However, wall thinning along the length of the pipes was examined, and wear was relatively uniform along the length of the sections. With no measurable flange-induced wear, the 10 wall thickness readings taken along the length of each HDPE section were averaged to obtain an average wall thickness value for each of the eight circumferential locations for each of the 11 sets of data. Table 3 lists the wall thickness measurements at the beginning and at the end of the test for each HDPE section and each location around the pipe. Figure 3 is a plot of the average of the Table 3 data, and shows the location and extent of wear.

A plot of wall thickness versus time for Location 1 (Figure 4) shows that wall thickness was not less than the manufacturer's guaranteed thickness of 0.969 inch until the December 7, 1985, reading. However, over 65 percent of the material handled during the test (1.7 million cubic yards) had been pumped by that time. From Table 3 and Figure 3, it is obvious that wear around the pipe was not uniform. Also, for all locations, wear was greater in Section 1. Slurry flow entered the HDPE pipes through Section 1 and then flowed into Section 2. It is possible that the uniform, smooth interior of HDPE Section 1 reduced turbulence enough to cause less wear in HPDE Section 2. Locations of greatest wear (7,0) and least wear (4,5) are the same in both sections. In both sections more wear occurred in the adjacent locations of 7,0,1 and 2. However, as seen in Table 2, only 38.4 percent of the total flow passed through the HDPE sections when these locations were on the bottom. This relationship is illustrated by Figure 5, which is a plot of the percent of the dredged material that passed through the pipe when each of the locations was on the bottom, and the percent of wear for each location. If the amount of material was the major cause of wear, then the location of greatest wear would be the location that passed the greatest amount of material. This is not what the data show.

Table 3. HDPE Pipe Wall Thickness Loss

Location on Pipe	Average Thickness Start of Test Inches	Average Thickness End of Test Inches	Difference Inches	Wear Percent of Original Thickness
Section 1				
0	0.993	0.800	0.193	19.4
1	0.979	0.890	0.089	9.1
2	0.991	0.872	0.119	12.0
3	1.013	0.931	0.082	8.1
4	1.034	0.995	0.039	3.8
5	1.057	1.036	0.021	2.0
6	1.058	0.969	0.089	8.4
7	1.028	0.718	0.310	30.2
Section 2				
0	1.018	0.897	0.121	11.9
1	1.050	0.955	0.095	9.0
2	1.056	0.961	0.095	9.0
3	1.041	0.967	0.074	7.1
4	1.018	0.981	0.037	3.6
5	0.995	0.993	0.002	0.2
6	0.983	0.924	0.059	6.0
7	0.981	0.692	0.289	29.5

Before conclusions could be drawn, it was necessary to verify the data and the accuracy of the wall-thickness gage. A second set of final readings was taken on April 22, 1986. These two sets of data were comparable and verified the highest percent of wear in Location 7. The calibration of the wall thickness meter was also checked and found to be correct.

During the course of the dredging season, the interior of the pipe was visually inspected by WES Hydraulics Laboratory personnel on three occasions, September 10 and October 19, 1985, and April 21, 1986. In September and October, the interior was smooth and unmarred with the finish going from highly glossy to a smooth satin. By April, the interior was marred with small, smooth nicks. The marring was uniform along the length of the pipe, but varied in intensity around the interior of the pipe, being most severe along Location 7 and least noticeable at Location 2. The butt weld bead (a result of the fused joint) was still intact in some locations but completely worn away in others (Figure 6). Also noticed on the final inspection was the appearance of small, smooth, circumferential ridges around the inside of the pipe about every 2 inches along the length of the pipe (Figure 7). This "washboard" effect has been observed in polyethylene pipe, but no definite cause could be identified by the manufacturer's engineer. It was suggested that this might be a function of material creep and the fact that flow was always in the same direction.

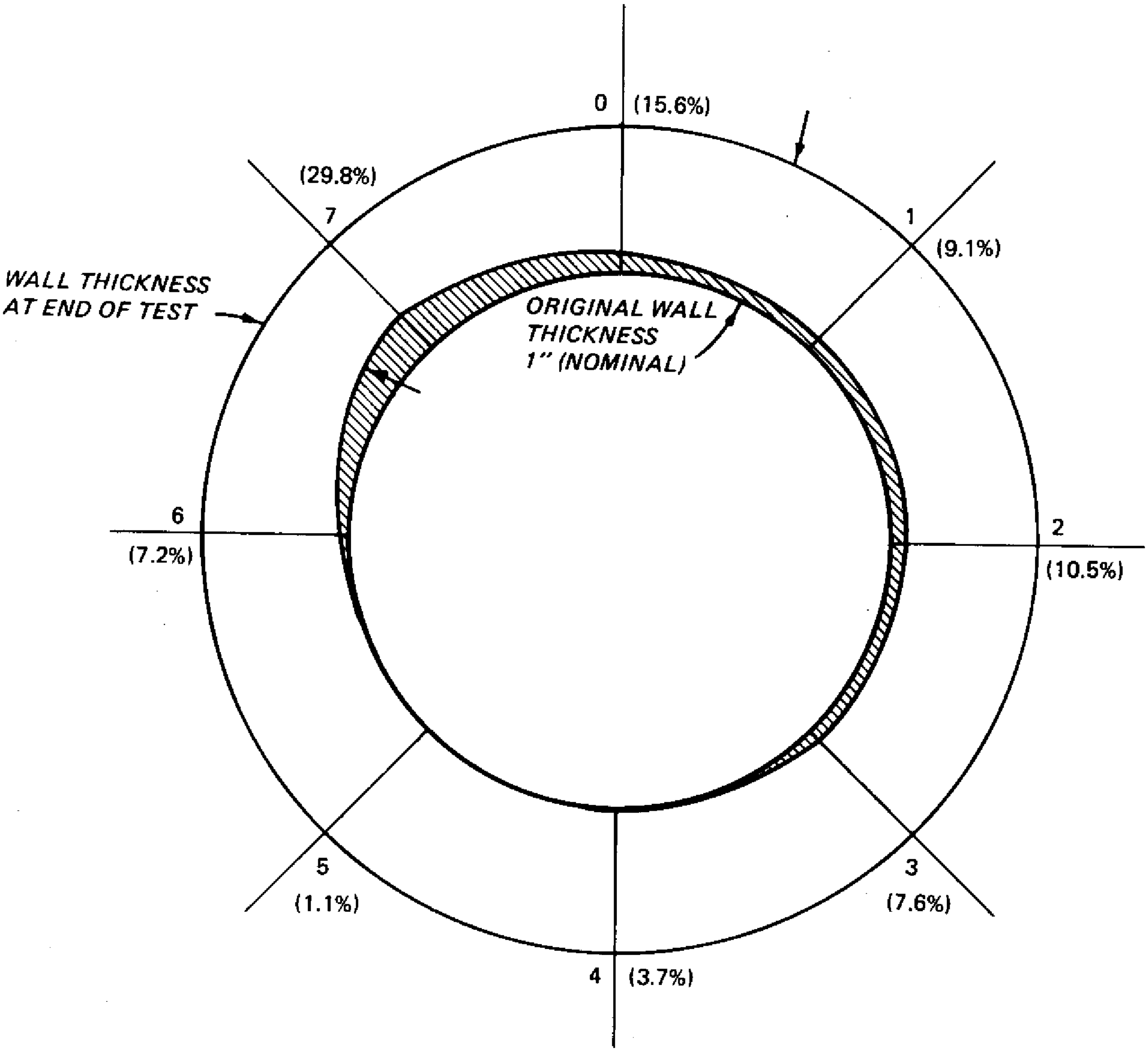


Figure 3. HDPE pipe circumferential wear pattern

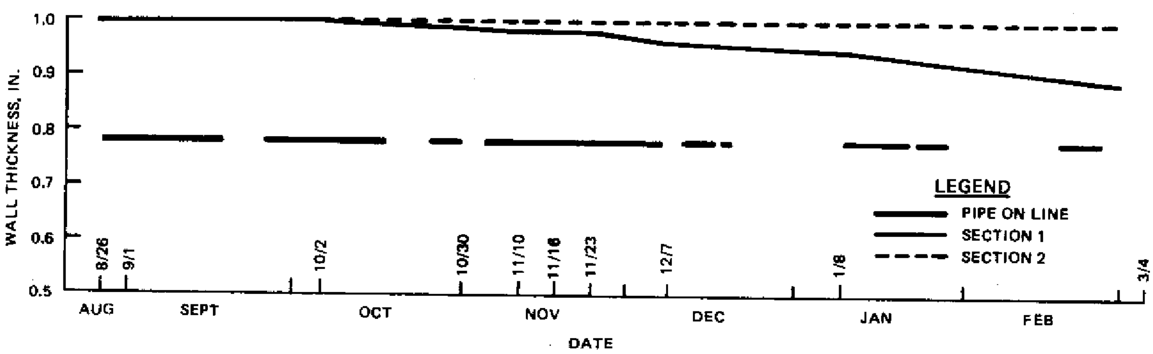


Figure 4. Location 1 - Pipe wall thickness versus time

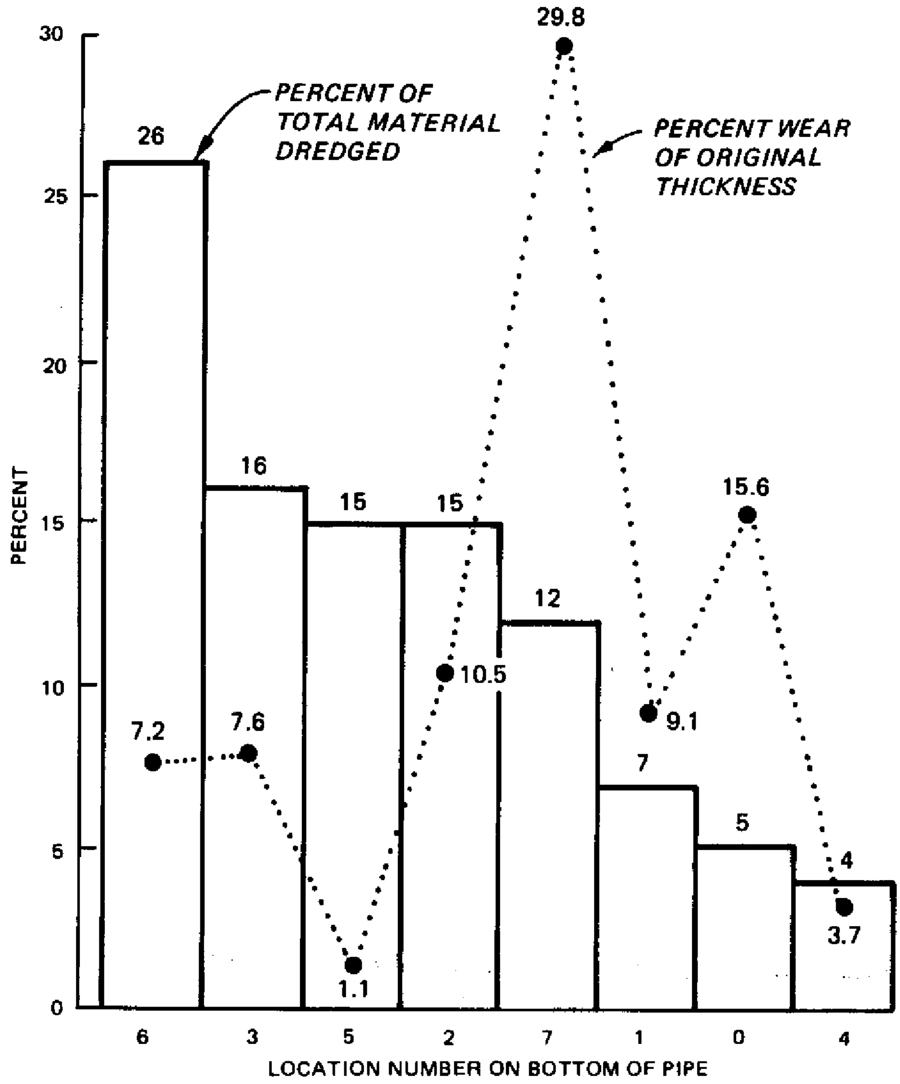
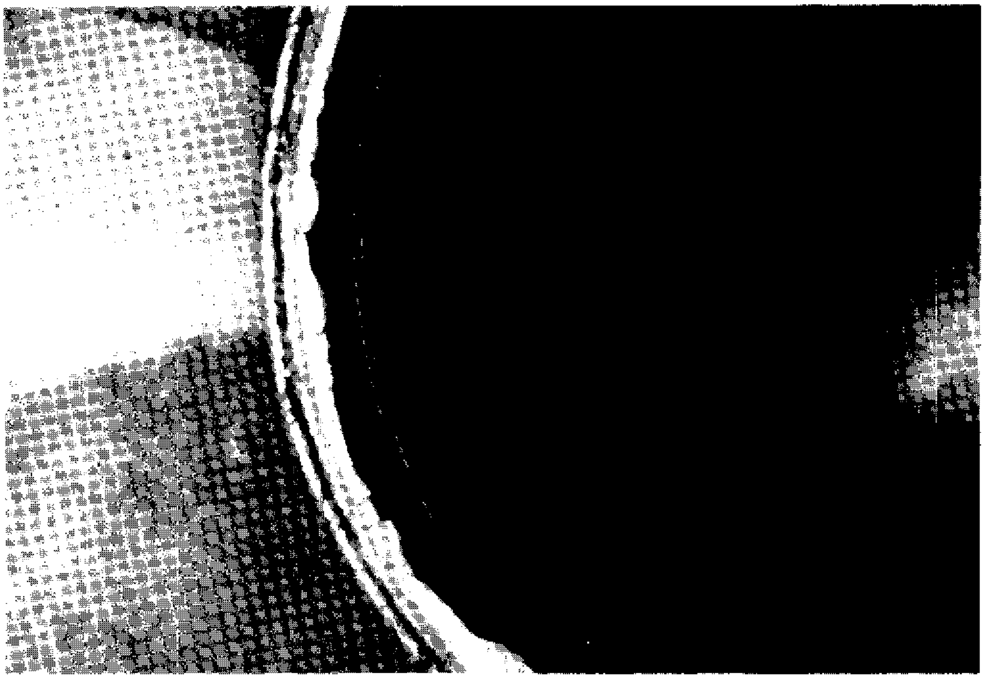
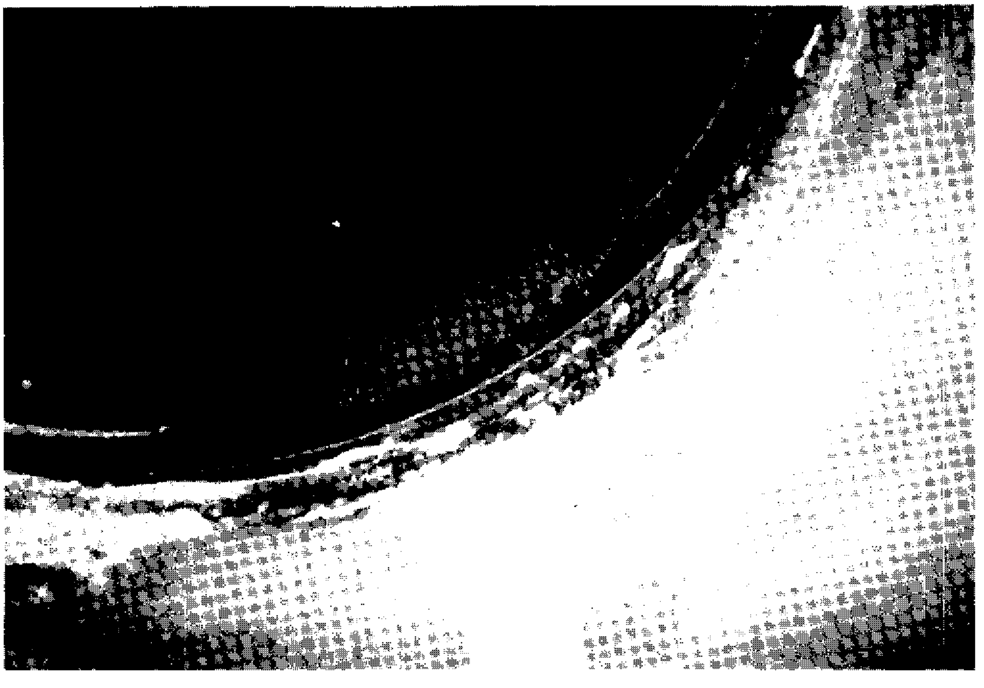


Figure 5. Percent of pipe wear compared with the percent of dredged material pumped



a. New pipe with smooth interior and perfect butt weld bead



b. After 2.6 million cu yds of dredging, interior is marred and butt weld bead is worn away in some locations.

Figure 6. Condition of pipe before and after dredging

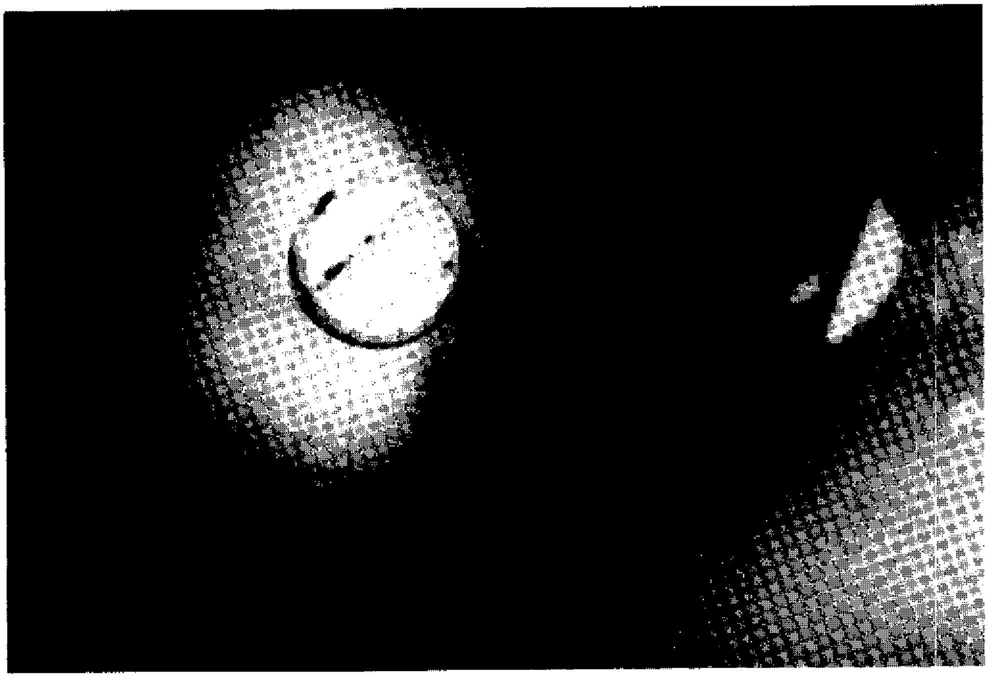


Figure 7. "Washboard" appearance inside HDPE section at end of test

Since wear was uniform along the length of the pipe sections, but was neither uniform through time nor around the interior of the pipe, the records of the type and character of material being dredged were examined. It was recorded that during the time frame of January 15-20, when Location 7 was on the bottom, 2-inch smooth river rock and pea gravel made up about 10 percent of the slurry being dredged. The velocity necessary to transport this heavier material was between 20 and 26 fps. The increased velocity along with the larger sized material would be expected to cause increased wear in any type of dredge pipe, and the greater wear at Location 7 was attributed to this.

Test Conclusion

The purpose of this test was to determine how HDPE pipe performs in an abrasive field setting. The sands of the Columbia River and the velocities necessary to transport them can cause excessive wear on dredge pipe and equipment. It was not the intent of this test to evaluate different grades of polyethylene pipe or different manufacturers' products. Nor was it possible to compare new HDPE pipe to new steel pipe because new steel pipe was not available in the pipeline at the time.

The information obtained during the dredging season was sufficient to indicate the usefulness of large-diameter HDPE pipe in dredging. Wear, in this case, was more dependent upon dredged material type than upon time or volume of slurry pumped. The pipe was easily rotated since the pipeline was moved and deployed many times during the dredging season. Although long lines of butt-welded HDPE pipe are the most efficient application, the dredge operator was able to use the quick-connect joints and easily incorporate the HDPE pipe into the existing discharge line.

The sections of HDPE pipe will be used and monitored for as many dredging seasons as possible before failure. The type and cause of failure will be documented, and using information of the history of the performance of steel discharge line, an attempt will be made to estimate cost differences between steel and HDPE pipe usage.

HDPE Users' Views

Dredge operators have used HDPE pipe for many years. Most are very pleased with the material and, when either steel or HDPE pipe can be used, prefer HDPE to steel for a variety of reasons. An informal survey of some HDPE pipe users was conducted to gain, from their experiences, the helpful hints and cautions of HDPE pipe use. In general, these operators have been using HDPE pipe for 3-6 years and are still using most of the original pipe they purchased. Their dredging jobs are relatively small ($0.5 to $1 million) and consist of maintenance dredging of mud and fine sands. The dredge sizes ranged from 12 to 18 inches in diameter. Every respondent agreed that in the proper application, their HDPE pipe outlasted steel, for some by as much as five times. Some found that HDPE was slightly less expensive than steel, and its ease of handling made mobilization and demobilization faster and less expensive than steel, and its ease of handling made mobilization and demobilization faster and less expensive. Some felt that contracts have been won because of these reduced costs. All agreed that the lightweight fusible nature of HDPE pipe is an economic blessing. Working lengths of 500 to 1,500 feet of butt-fusion-joined HDPE are easily floated to the project site. As the pipe is filled with slurry, it sinks to the bottom and conforms to the contours of the terrain. Ease of repair was also cited as a favorable feature. Damaged sections of pipe can be removed with a chainsaw and repaired with a circle clamp or a butt-fusion joint. The need for elbows is reduced by the flexible property of the material which not only allows the pipe to conform to the terrain but allows bends of 20 to 40 pipe diameters. Fewer elbows and fused joints provide improved hydraulic efficiency which requires less pumping power.

Many dredging practices, such as pipe rotation, are common to all situations; however, some practices are specific for HDPE pipe. None of the users employ HDPE pipe in the suction line of the dredge, and all use steel immediately following the dredge pump. The rigid nature of steel and its higher working pressure are important in these locations, and this practice is based on intuition, tradition, and experience. The length of steel used before switching to HDPE varied from as little as 40 feet to as much as 4,000 feet. Again this is based on individual preference.

Depending on the job, it may be cost effective to rent rather than buy the butt fusion equipment. Pipe sections can also be joined using circle clamps, which should be 1.5 pipe diameters wide to distribute pressure and not deform the HDPE pipe.

The material is rugged, and nicks and gouges on the outside have not necessarily resulted in pipe failure; however, there is a need to teach field personnel to be careful when handling HDPE pipe with heavy equipment. As was seen during the field test reported in this paper, it is helpful to know the type of material that is to be dredged. Sharp material such as oyster shells can be dredged for a limited time; however, dredging of this material for more than 24 hours frequently results in tears and leaks. Coral and sharp volcanic sands are not suited to HDPE pipe.

The light weight of the pipe can create problems when anchoring submerged HDPE pipe in water deeper than about 15 feet or with currents greater than about 4 knots. If the slurry can be maintained at 30 percent solids, the pipe may remain safely submerged. Although HDPE pipe manuals explain proper anchoring procedures, several operators choose to use steel in these situations.

Conversations with one HDPE pipe manufacturer's engineer revealed that overpressuring a pipe is acceptable in some instances, but the wear life will be significantly reduced. Manufacturer's tests of pipe under given pressure and slurry velocity conditions produce a predicted wear life of 50 years. Changing the pressure and/or slurry velocity will correspondingly change this value. There are valid applicatons of HDPE pipe where wear life of the pipe will be reduced but will still be sufficient and economical for the project requirements. Properly sized pipe for the working pressures and velocities is always preferred. Overpressuring HDPE pipe does not change the material density; the shorter life span is a function of the long-term creep property of the material. This long-term creep (relaxation) results in a slow thinning of the pipe wall, which lowers the pipe pressure rating. HDPE material has the ability to "recover" from overpressuring if certain limits are not exceeded. A manufacturer's engineer is always available to provide guidance in this area, and answer any questions concerning the use of his product. In general, if quality pipe is purchased, and the manufacturer's suggestions and recommendations for use are followed, the project should proceed without HDPE problems.

No product or procedure is without some failures, and the use of HDPE pipe in dredging is no exception. Most failures can be evaluated and the results added to the pool of HDPE pipe information. One maintenance dredging project reported very disappointing results almost immediately after installing HDPE pipe. The recommended high-pressure pipe was not purchased, and 12-inch nominal, size low pressure (50-psi), 40-foot-long sections flanged on both ends were used. The material dredged consisted of angular 1/4- to 1/2-inch diameter sand, and gravel with some 6-inch-diameter rocks. A working pressure of 60 psi was maintained with a pumping velocity of 15 fps and a 12-percent solids concentration. The first failure was located 2 to 3 feet from the flanged end of a pipe section and occurred within 3 days. At first, the dredger was surprised at the smoothness of the slurry flow; but within 24 hours he was aware of a dramatic increase of pipe resistance. When the pipe failed, the interior walls were examined and found to be badly torn with shreds of material loosened from the pipe wall. It was the practice of this dredge operator to rotate the dredge pipe 1/4 turn every 2 days.

A combination of factors resulted in this poor performance, which could be attributed to misapplication rather than to poor quality material. Slight overpressure, sharp dredged material, and many flanges would not have damaged a steel pipeline. In this application, the additional turbulence induced by the flanges and the sharpness of the material quickly eroded the pipe. Fused connections are recommended in HDPE pipe to reduce slurry turbulence and pipe wear. There was also evidence of poor pipe alignment which accentuated the effects of the turbulence.

This operator agreed that perhaps higher pressure rated pipe would have lasted longer. He also agreed that HDPE is not well suited to dredged material that includes gravel and rocks. He did remark that the HDPE pipe was considerably easier to handle and install and conformed easily to the uneven terrain. He does not plan to use HDPE in similar dredging situations but would consider using it if the dredged material was finer.

Another dredge operator felt he had received poor quality pipe or butt fusion equipment when his HDPE pipe experienced failure at the stubb end fusion joint. The operator was not discouraged by this experience; he is more careful in his selection of pipe manufacturer and continues to use HDPE pipe.

Within the dredging community, certain pipe companies have the reputation of producing a quality product while others have earned the distinction of producing a product which does not perform well. A. manufacturer's reputation and guarantee policy should be considered when purchasing HDPE pipe.

Summary

As the dredging industry expands, accompanied by higher operating costs and increased distances to disposal sites, new technology and materials are being introduced. HDPE pipe can be an efficient alternative or supplement to steel discharge lines. The physical and mechanical properties of HDPE are sufficiently different from steel that it must not be treated as a rigid pipe. Its flexible, lightweight, abrasion-resistant nature offers the dredge operator new freedom in pipeline design, life, cost, and maintenance.

The previous discussion has identified many of the HDPE characteristics that make it well suited to dredging situations. Manufacturer's recommendations should always be followed concerning the pipe specifications for a specific project. Other practices, proven by experience, should be seriously considered such as the elimination of flanges by using the butt weld joint, rotation of pipe to distribute wear patterns, and the avoidance of extensive dredging of sharp, angular material or large rocks. However, the use of HDPE pipe gives the dredge operator certain "liberties" that can be employed such as slight temporary overpressuring, that will, in the long term, prove economical in cost and time for the project.

The Portland HDPE field test proved that large-diameter HDPE pipe can be successfully and easily incorporated into an existing dredge discharge pipeline. After the pipe passed over 2.6 million cubic yards of coarse Columbia River sand, it was determined that wear was more related to the type of material than the amount of material dredged.

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Biodata

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Integrated Dredging and Processing of Alluvial Mineral Deposits

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Abstract

Alluvial deposits, commonly termed "placers," are a naturally liberated resource of precious metals, gemstones and so-called black sands containing heavy minerals such as magnetite, ilmenite, chromite, zircon, monazite and rutile. The inland deposits have long been the target of mining and dredging operations. However, with the steady depletion of potentially economic resources, more efforts are now being applied toward exploration and exploitation techniques in a marine environment above (beach deposits) or just below (offshore deposits) the current sea level.

This paper reviews the generic deposition and characteristic properties of alluvial mineral deposits. It highlights their recovery techniques, recent technological developments and integrated mineral dredging and processing practices in a marine environment. Also presented is an insight into the development potential of Nova Scotia offshore gold deposits.

Characterization of Alluvial Minerals

The alluvial minerals are the product of a complex weathering and erosion process of the outcropping rocks. The naturally liberated mineral-rich sediment may accumulate near the source rock or enter into a dynamic system of redistribution, gravity sorting and deposition in environmentally favorable settings. Concentrations of alluvial minerals are found in various amounts along the river valleys, beaches, dunes, in estuary basins and in offshore drowned systems covered with sediments, as well as deposited in bedrock crevices, pits and fractures. The concentrated layers of heavy minerals may have thicknesses of several millimeters to several meters depending on the deposition volume and its composition. Since the alluvial mineral deposits vary in degree of isolation, size, age and composition, separate studies are required to assess the economic value and the future development potential of each deposit (Oduntan, 1984; Hale and McLaren, 1984; Chaziteodorou, 1977).

From the commercial interest value, the alluvial deposits are classified into two groups: (1) relatively large volume and low unit value commodities, e.g. sands and gravels, and (2) low grade, high value commodities, e.g. noble metals, precious stones and so-called black sands with magnetite, ilmenite, rutile, zircon, monazite, chromite, cassiterite, etc. In the former group all or most of the volume mined is utilized, whereas in the latter group only a small fraction containing the valuable minerals is utilized. This latter group, or the placer minerals, is considered in this paper.

The economic importance of the placer minerals is exemplified in their many uses and applications (Table 1) for which there are limited or no substitutes. The value of zircon, rutile and scheelite have increased more than 1.5 times, and gold and platinum more than 6 times, between 1958 and 1980 (Gomes and Martinez, 1981). The particle size of the placer minerals ranges from a few microns up to coarse nuggets and crystals (Macdonald, 1983). The main characteristic properties of some placer minerals is shown in Table 2. The differences in particle size, shape, specific gravity, magnetic and electric properties, surface wetability and other physical properties are employed in mineral processing techniques to separate the valuable minerals from each other and from the gangue minerals.

Dredge Mining and Developments

The exploitation of placer deposits has long attracted the interest of mining and dredging operators. When the supply of water is sufficient, dredge mining proved to be more efficient than other mining systems (Woodsend, 1984; IHC, 1983). Table 3 presents a comparison between dredge mining and other placer mining systems.

After an alluvial deposit has been explored and an economic appraisal has been made, the selection of a dredge mining system depends on the analysis of several factors, such as: (1) the geological and topographical characteristics of the deposit, (2) the digging conditions and restraints of the marine environment, (3) the technological limitation of the dredging system.

Table 1. Main Applications of Some Placer Minerals

Mineral	Metal	Applications
Ilmenite	Titanium	Aerospace industry, high strength and corrosion resistence, oxide form used in pigments
Platinum Group		High temperature applications, catalyst, jewelry, dental alloy
Zircon	Zirconium	Refractories, ceramics, abrasives, chemicals
Chromite	Chromium	Increases hardness and toughness of steel, electroplating, refractory pigments, chemicals
Cassiterite	Tin	Plating, bearing metals, bronze, solders
Gold	Gold	Jewelry, currency, electronics, dentistry, plating
Monazite	Thorium	Radioactive metal

Table 2. Characteristic Properties of Valuable Placer Minerals

Mineral Valuable	Percent Density	Relative (Mhos)	Hardness Property	Magnetic Property	Electric
Gold	Au	15.6 - 19.3	2.8	NM	C
Platinum	Pt	17.0	4.5	NM	C
Monazite	9 Th O	4.9 - 5.3	5.0 - 5.5	M	C
Chromite	46.2 Cr	4.3 - 4.6	5.5	WM	C
Ilmenite	31.6 Ti	4.5 - 5.0	5.0 - 6.0	M	C
Rutile	60.0 Ti	4.2	6.0 - 6.5	NM	C
Cassiterite	78.8 Sn	6.8 - 7.1	6.0 - 7.0	NM	C
Zircon	67.2 ZrO	4.2 - 4.7	7.5	WM	NC
Diamond		3.5	10.0	NM	C
Scheelite	63.9 W	5.9 - 6.1	4.5 - 5.0	NM	NC
Magnetite	72.4 Fe	5.2 - 5.6	5.5	M	C

*compiled from Weiss (1985)
M–Magnetic; WM–Weakly Magnetic; NM–Non Magnetic; C–Conductor; NC–Non Conductor

Table 3. Parameters of Different Mining/Processing Systems*

System	Excavation	Transport	Mineral Treatment	m^3 Water / m^3 Ore	kwh / m^3 Ore	m^3 Ore / man hour
1	monitor	hydr. lift	sluice box	30	15	1
2	monitor	gravel pump	conv. jig plant	14	13	1,3
3	shovel	belt. conv.	do.	8	8	1,2
4	shovel	dump truck	do.	8	8	1,1
5	dredge mining	integrated	IHC jig plant	6	2	20

* IHC Holland (1983)

The volume and grade of the deposit determines the production capacity of the operation. Macdonald (1983) estimated a deposit volume of 10 million m^3 for a small-scale operation and at least 120 million m^3 for a large operation based on the capital investment costs of the bucket ladder dredge in 1980. Koesmadi et al. (1975) suggested that prior to planning an offshore dredging operation at least 10 years' life of the reserve must be proven.

Bucket ladder dredges and cutter suction dredges are the main dredge types used in mining placer deposits. Of the other alternatives, clamshells, dragheads and jet lifts have a limited scope for mining deposits of loose, uncohesive alluvium (Macdonald, 1983; Cross, 1979; Herbich, 1978; and others).

The bucket ladder dredges are preferred when mining tight and hard formations having clay lenses and bedrock outcrops, whereas the cutter suction dredge is used mostly for civil applications and overburden removal (IHC Holland, 1983). Some of the rationale for this preference is based on the following:

- The bucket exerts mainly an upward shear force on the bank, while the blades of the cutter work against a resistance in compression, thus increasing the power consumption.
- The cutterhead digs efficiently only in its undercutting swing, whereas the bucket ladder dredge is digging in both swing directions.
- Power and maintenance costs incresases with hydraulic dredges when digging in compact, coarse, more abrasive and high density mineral deposits. The hydraulic dredges tend to produce turbidity and a hydraulic classification of the material around the suction inlet, e.g. heavy gold particles settle out faster and fines may be floated away by the turbidity, while the sands and gravel are pulled into the suction.
- It is more difficult to maintain a constant slurry density feed to the processing plant from cutter suction dredges than from bucket ladder dredges. Hydraulic systems often produce quantities of water in excess of that needed for the processing operations.
- Plugging of the cutter or pump with buried fibrous roots and timber creates disruptions to the operation.

A bucket ladder dredge system presents some problems also, such as:

- Losses of valuable minerals from the bucket during the discharge into the drop chute at the top tumbler. Save-all devices must be installed to catch the spilled material.
- The initial capital costs and maintenance costs of a bucket ladder dredge are higher than a hydraulic dredge.
- The maximum dredging depth is limited, compared to hydraulic dredges.
- A bucket ladder dredge usually requires more personnel than other types of dredges.

Mechanical components and electronic control equipment, i.e. dredge profile monitors, drive systems, mooring techniques, processing plant systems and equipment, and tailings discharge systems have all been developed or evolved towards higher dredge performance, deeper dredging and greater economic recoveries.

A new concept in bucket chain design increased the bucket capacity to a maximum of $0.88m^3$ of water depth (Donkers, 1980). For easy handling, transportation and lower maintenance costs, modular dredging units have been recently marketed by IHC Holland as pump unit, cutter unit and main and side pontoon modules.

To combine advantages from the bucket ladder and hydraulic dredges, a new type of bucket was designed (Figure 1) for the conventional bucket wheel dredge (IHC Holland, 1983). The characteristics of four commercial types of IHC bucket wheel dredge are presented in Table 4, and the schematic features in Figure 1.

Table 4. Characteristics of IHC Dredging Wheels

Wheel	750*	1500*	2900*	4000*
Wheel Power (kW)	75	170	320	550
Pump Power (kW)	370	810	1470	2060
Dia. Suction Tube (mm)	400	550	650	800
Dredging Depth (m)	10	14	16	16

*Model number indicates totals machinery output in hp

Some of the main features of the IHC bucket wheel are:

- The bottom and curved back of the conventional wheel bucket have been completely omitted to avoid accumulation of the material and plugging of the suction passage.
- The outer edge of the bucket is specially shaped to guide the material into the suction inlet.
- The space between the buckets is smaller than the size of the material entering, therefore preventing obstruction of the suction and discharge line or the pump.

The quest for the economic dredging of mineral deposits has led to the improvement of the existing dredges for use in specific local conditions (Dieperink, 1978; Anon, 1981 and 1982; Lim Che Wan, 1983; Houston, 1983; Woodsend, 1984; Lewis, 1984). New concepts have also been proposed for deeper dredging, such as continuous-dragline dredge and a multi-grab dredge (Hewitt, 1978).

Recovery Techniques and Developments

The processing of minerals evolved as a necessity to recover the valuable minerals from a mined deposit and produce a required marketable concentrate. The recovery operation depends on the amenability of the minerals to a treatment process and it is based on the differences in the physical properties of the minerals, such as particle size, shape, specific gravity, magnetic and electric properties, surface wetability and other properties.

Screening and gravity separation are the most common methods to recover placer gold and heavy minerals from alluvial deposits, whereas the magnetic and electrostatic separations are used mainly in cleaning processes and to separate specific heavy minerals from each other. The differences in surface wetability of the minerals is applied in the flotation process to recover fine and ultra fine particles.

Early inland dredge mining operations used sluices and tables as gravity concentration devices to recover gold from sand and gravel deposits. With declining grades, Humphreys spirals and various types of jigs of higher treatment capacity were introduced.

Technological developments in the processing of low-grade minerals has led to more efficient systems and equipment which could also be applicable to the recovery of marine placer minerals. Major acheivements were made in primary gravity concentrators for mineral sands in Australia (White, 1984). The Wright Impact Plate Separators proved efficient for the recovery of beach minerals from several black sand deposits in Australia and West Africa (Macdonald, 1983). The Wright Impact Plate separators differ from other types of sluices by the presence of an impact plate which divides the slurry flow into two streams. The angle of the impact relative to the slurry flow determines the recovery rate.

Mineral Deposits Ltd., Australia, developed double cone concentrators to improve the efficiency of the early Reichert cone systems. The cones have an average treating capacity of 80 t/h and can achieve separations of particles as fine as 40 microns (Burt, 1984). Sierra Rutile Ltd. has recently been using the cones separation system on its dredging operations to recover rutile deposits from along the Sierra Leone coast, West Africa (Anon, 1981). Reichert cones have also been used by dredging operations in New Zealand (Buist et al., 1978) and at Richards Bay, South Africa (Anon, 1976).

In an effort to improve the recovery of fine gold and other heavy minerals and to increase the treatment capacity of the conventional jig cell units, IHC Holland developed the radial jig, originally patented in 1967 as the circular jig by N. Cleveland (IHC Holland, 1983). Unlike the conventional jigs, the IHC radial jig works without backwater addition or with a limited amount. The IHC jig uses a specific drive system to induce a fast upstroke and a slow downstroke wave pattern, "saw tooth" wave, rather than a harmonic wave as in a conventional jig (Figure 2). The short, sharp pulsation minimizes the fines loss and allows their recovery during longer, gentle suction.

To overcome the difficulties in handling and transportation of the standard circular jigs (up to 7.5m in diameter), a module-jig concept was developed. The treatment capacity of a module ranges from approximately 40 to 83 m^3/h solids and can be assembled in any number, twelve modules being required for a complete circle. The IHC radial jigs have been applied recently by some offshore dredging operations in South East Asia for the recovery of cassiterite (Kramer, 1984), in Alaska for the recovery of gold (Anon, 1985), in Brazil and other countries (IHC Holland, 1985). Figure 3 presents a comparison between the IHC radial jig, conventional jig, tables and riffled sluice for the recovery of fine gold particles.

The recovery efficiency of various types of hydrocyclones and centrifugal separators on dredge plants have been investigated by the Soviets (Anon, 1981). The wide angle concentrating hydrocyclone reported the highest recovery of fine gold, approximately 90 percent for gold particles in the range of 0.05mm.

Several other gravity concentrating devices have been developed or evolved from existing ones to produce higher recoveries of fines and enrichment ratios; however, their performance on offshore dredge plants has been limited or not investigated (Rockwell, 1985).

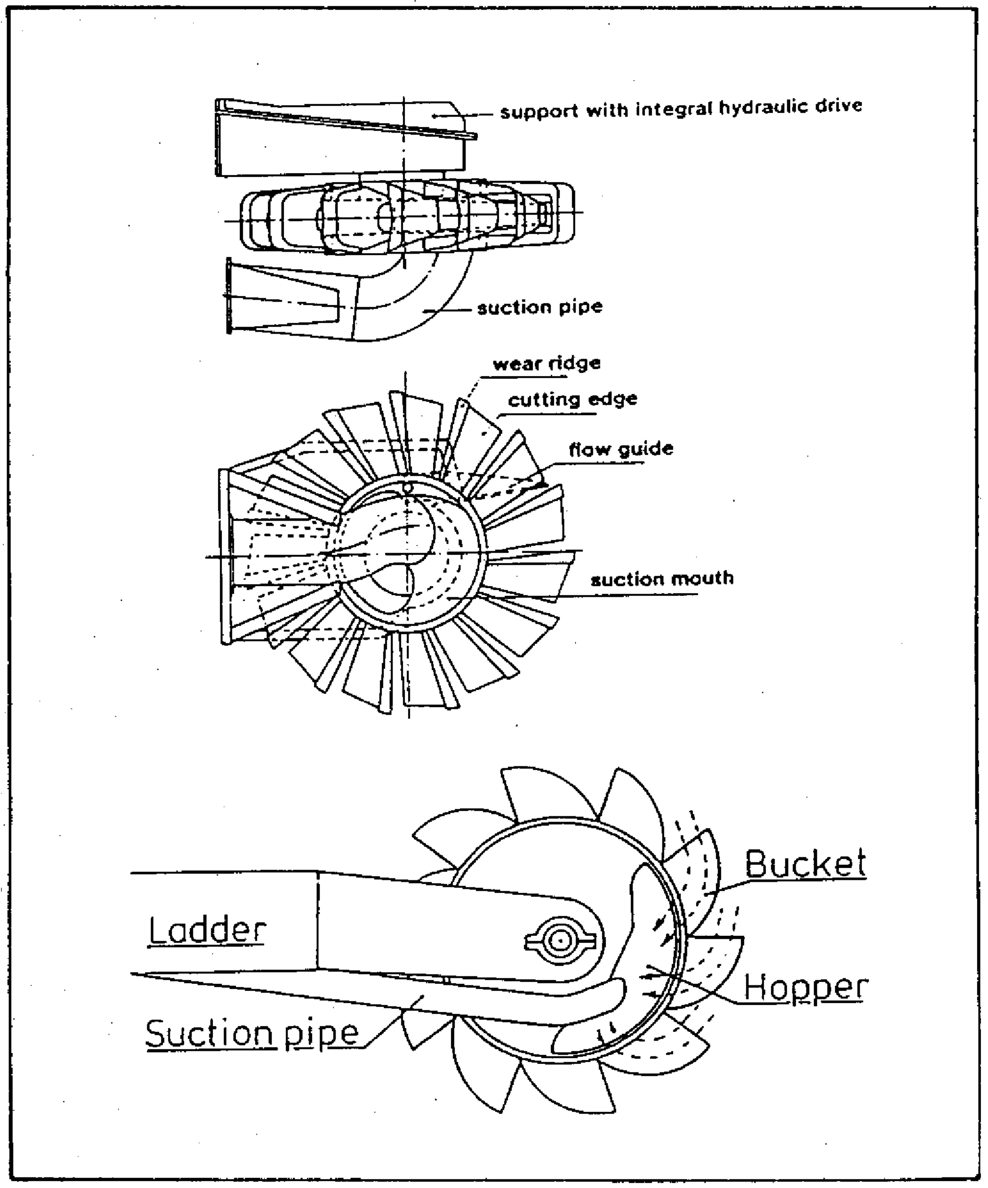

Figure 1. Schematic features of Conventional Bucket Wheel (lower) and IHC DredgingWheel (upper)

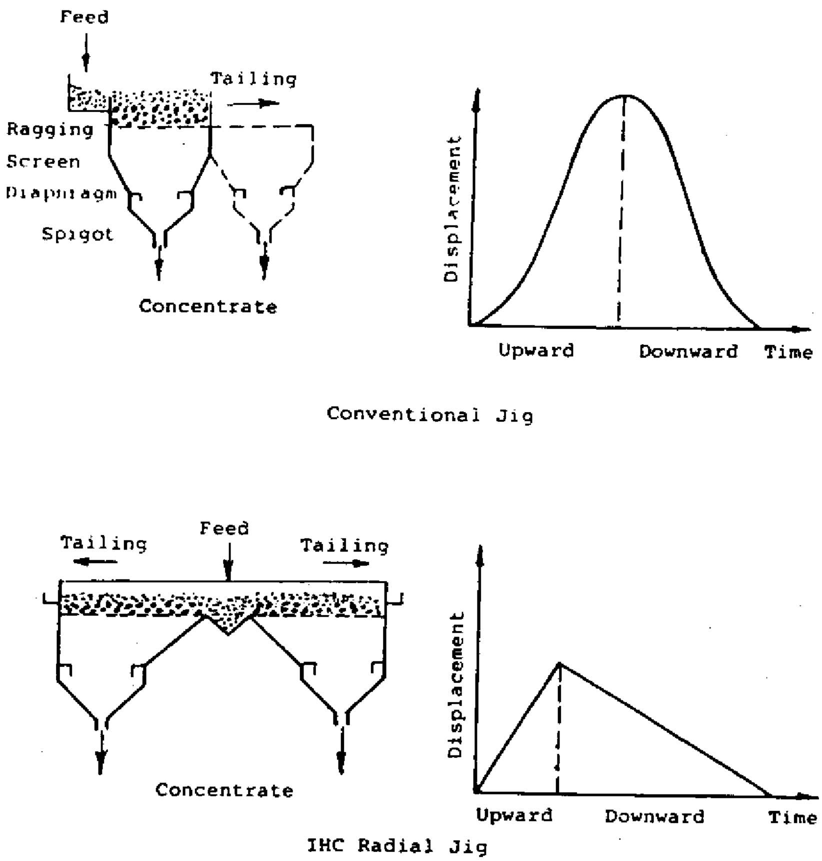

Figure 2. Wave pattern comparison between a Conventional Jig (upper) and IHC Radial Jig (lower)

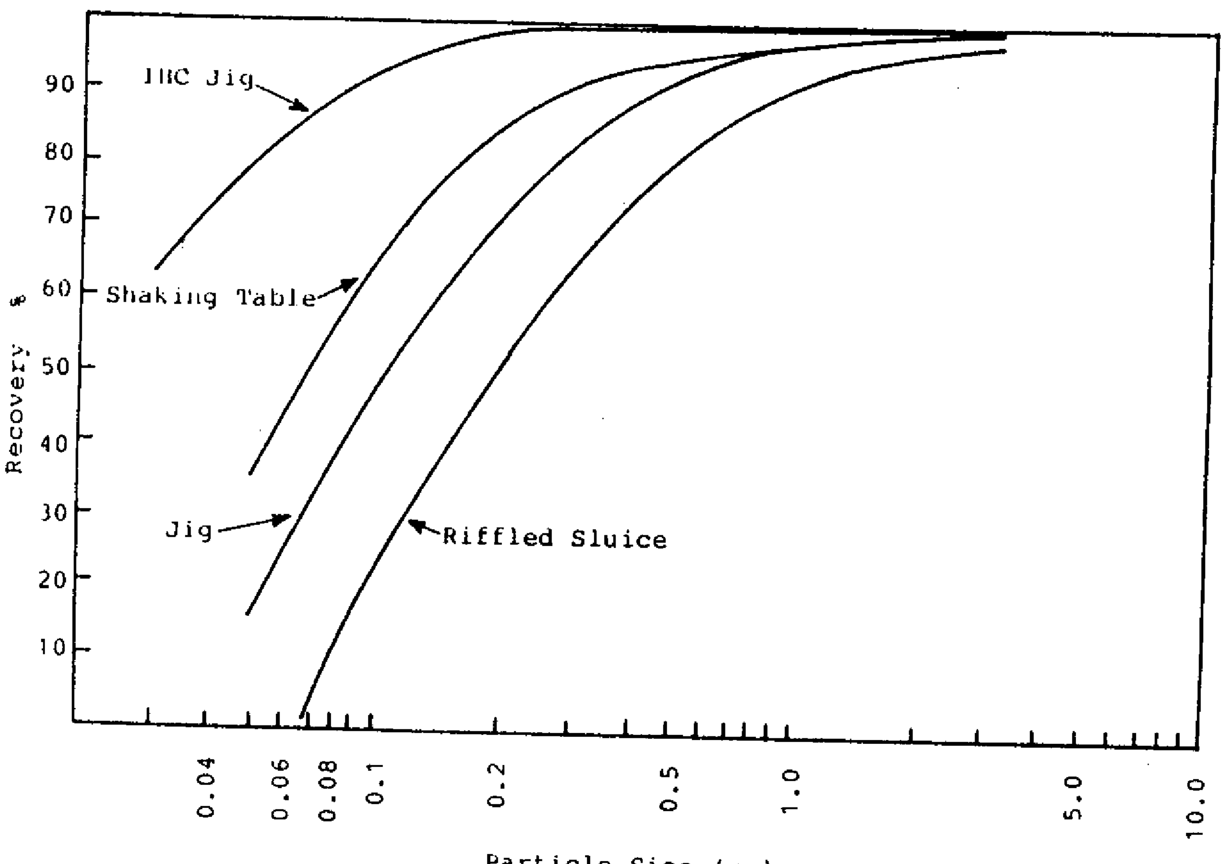

Figure 3. Gold recovery comparison of various concentrators

Recovery Practices at Sea

The processing of placer minerals at sea has the advantage of reducing the need for the transportation of large quantities of waste minerals which can be rejected on site. Offshore dredging operations in the exposed waters of South East Asia have the processing plant on board to add weight and to increase the stability of the dredge at sea and also to reduce the operating cost by integrating the services (Koesmadi et al., 1975; Donkers, 1980; Lim Che Wan, 1983). In coastal and protected waters, the processing plant is on a floating platform moored independently of the dredge and lying behind it. This arrangement is commonly used with some of the hydraulic types of dredges to increase the mobility of the dredge. The dredged material is pumped through a floating pipeline to the plant for processing (Anon, 1976; Buist et al., 1978; IHC, 1983; Kramer, 1984).

Compared with land-based operations, processing at sea imposes restrictions due to the limited availability of plant space, electric power and other facilities. On modern integrated dredging/processing operations, the processing plant consists of two major sections: the feed preparation and a concentration section which has a number of jigs or cones. The operation produces a bulk concentrate at a high recovery and a reject containing sands, gravels and clay minerals. The bulk concentrate is shipped to shore for cleaning and upgrading. This may take the form of further gravity concentration, magnetic and electrostatic separation, heavy media techniques or flotation.

In the feed preparation section of the plant the dredged material is disintegrated, sized, deslimed and the density of the slurry adjusted. One or two revolving screens perform the function of scrubbing and sizing. Spray water jets, under pressure inside the revolving screen disintegrate the lumpy material and clay. The oversize material, usually coarser than 16mm, is disposed of. The undersize is distributed to the first rougher concentrators. The rougher concentrate is passed through several cleaner and recleaner stages before a bulk concentrate is obtained. The reject from the rougher concentrators is disposed as final tailings and the rejects from the cleaner and recleaner concentrators are recirculated.

Any inefficiency in the recovery process could result in losses of valuable minerals, in the rejected tailings, at sea. This may reflect on the returns of a dredging operation. Dankers (1980) noted that most of the inefficiencies of a processing system are due to an underdesigned plant capacity to treat feed fluctuations from the dredge and their recirculated products.

For particular cases when a placer deposit contains minerals in an oxidized state, partially liberated, or as fine and ultra fine particles, processing at sea becomes more complex and implies extensive studies prior to defining a suitable processing system. In addition, the concentrators operating in the marine environment should demonstrate the following:

- tolerate fluctuations in feed rate, slurry density and particle size.
- tolerate moderate sea motions and vibrations from the machinery on board.
- operate with minimum supervision and maintenance.
- produce high recovery and enrichment ratios with a minimum power consumption.
- produce environmentally safe wastes.
- to be accessible at relatively low cost.

Since each placer deposit has distinctive geological and mineralogical characteristics and each equipment performs differently when the working conditions changes, preliminary studies and testwork are required before the conception of the final design of a dredging/processing operation.

Nova Scotia Offshore Placer Gold Deposits

Gold has been mined from inland and beach deposits within the province of Nova Scotia since 1862. Early recoveries of gold were reported from beach deposits at Isaac's Harbor, Wine Harbor, Tangier Harbor, Gold River and the Ovens. Placer mining, however, never assumed an important proportion after the turn of the century (Malcolm, 1976). There are approximately 190 gold occurrences in the province (Figure 4) categorized as being either load, paleoplacer or placer (Ponsford and Lyttle, 1984; Fowler and Miller, 1985). Most of the gold occurrences are located in the southern mainland of the province and are mainly confined to the metasedimentary rocks of the Meguma Group, comprised of quartzites, greywacks, slates and granitic intrusions. The Group underlays 30 percent of the southeastern part of the province and extends for approximately 40 km offshore, covering an offshore area of 26,550 km^2 (King and MacLean, 1976).

The rich, gold-bearing rocks of the Meguma Group provide a suitable source and the processes of glacial erosion, fluvial transportation and subsequent transgresssions of the sea have been favorable for the formation of marine placer deposits. Sediments supplied to the Shelf were estimated by Hopkins (1985) to contain up to 0.06 oz/m^3 of gold and an unidentified quantity of cassiterite, scheelite, ilmenite, garnets and zircon.

During the retreat of the ice and isostatic rebound age, the major mainland rivers extended valleys over the rocks of the Meguma Group to approximately 25 km off the coast (Samson, 1984). The subsequent transgressions of the sea reworked and redistributed the sediments. Some post-glacial concentrations of heavy minerals have been preserved and covered by silts and clays. The possibility also exists that alluvium from the pre-glacial channels and streams survived the glaciation. Such deposits would now be buried under the post-glacial sediments and tills. Conceptual models of possible types and sub-types of offshore placer gold deposits in the region have been published by Samson (1984), and Hopkins (1985), and continues to be a subject of investigation (Hale, 1985; Fowler and Miller, 1985).

The major commercial interest in Nova Scotia offshore placer gold commenced in 1968 with an extensive exploration program by Matachewan Canadian Gold Ltd. The survey focused primarily on the areas of submerged river valleys on the southeastern shore of the province. Promising potential areas of gold-bearing sediments were identified at Country Harbor and around the Ovens Peninsula within approximately 8 km from shore.

A suction dredge was constructed by Matachewan in 1969 and some gold was recovered from the Ovens area. The volume of an outlined area in water depth of less than 15m at the Ovens was estimated at 4.6×10^6 m^3 with values up to 0.15 oz/m^3 gold (Libby, 1969). However, due to the low price of gold at that time ($39.50 US/oz) and other technical difficulties, the dredging operation was discontinued and the claims were lapsed in 1970. Seabright Resources Ltd. in 1980 reinstituted the claims and are still investigating their potential.

The Country Harbor area has attracted the most interest because of its surrounding districts of gold mining history and future potential. From 1862 to 1974 the gold districts in the are contributed approximately 50 percent to the total gold production of the province (Hopkins, 1985).

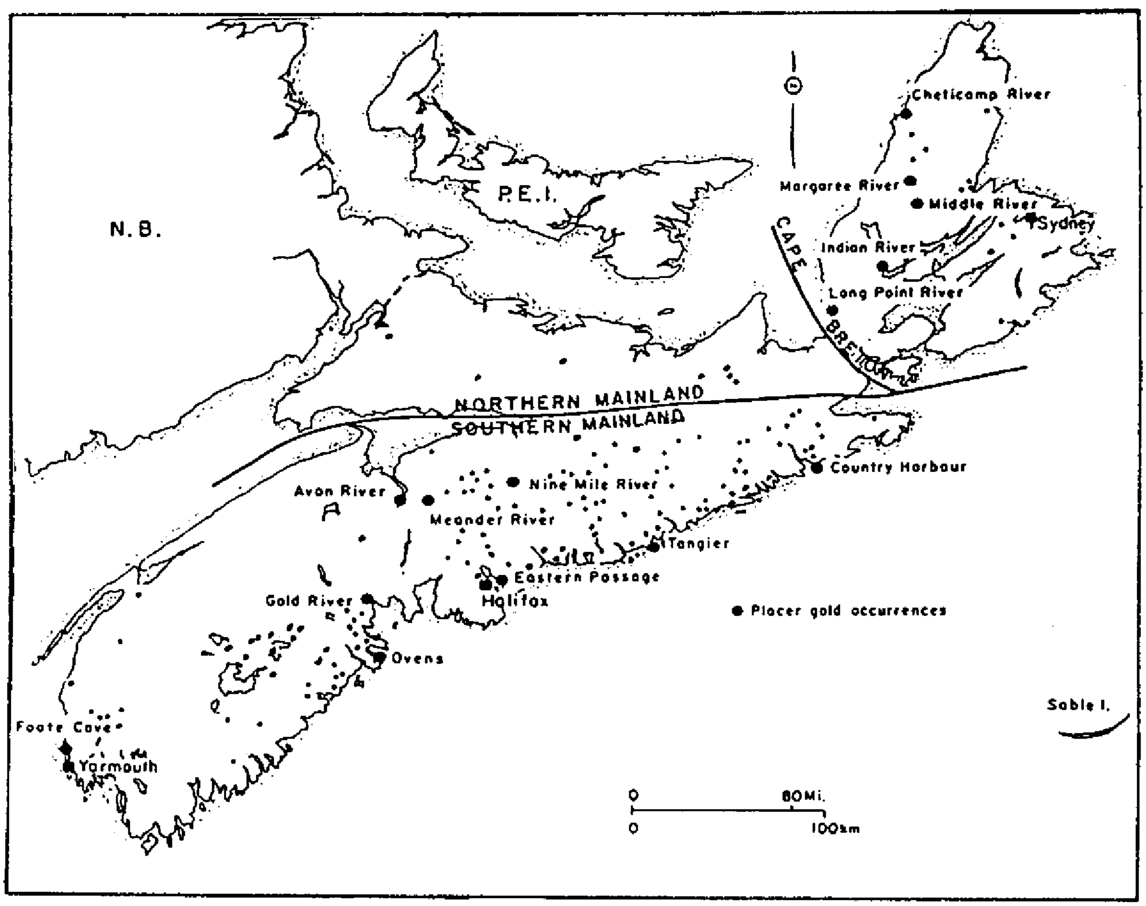

Figure 4. Nova Scotia Gold Occurrence

Matachewan Canadian Gold Ltd. outlined in 1968 approximately 28 x 10^6 m^3 of gold-bearing sediments offshore Country Harbor in water depths of less than 30m. Seabright Resources Ltd. reinvestigated the target sites in 1980 and located additional auriferous sediments. The gold concentration in the sampled sediments were reported to occur mostly in the -63 microns fraction (Figure 5). The highest content attained was 4250 ppb. (1.987 oz/m^3) and the lowest less than 30 ppb. (0.014 oz/m^3) gold (Hopkins, 1985; Coughlan, 1985).

Exploration of a smaller scale was conducted in the same area by Cities Services Minerals Corp. in 1968 at Wine Harbor and by Barrett (1981) at Isaac's Harbor. Cities Services Minerals Corp. estimated a volume of approximately 6 x 106 m^3 of auriferous sediments in the Wine Harbor Basin averaging 0.079 oz/m^3 gold (Samson, 1984).

The limited commercial offshore exploration suggests that individual deposits offshore Country Harbor could contain as much as 30 x 106 m^3 of gold-bearing sediments. Many of the outlined deposits are in relatively shallow water and would permit recovery by present dredging technology. Assuming a minimum average grade of 0.015 oz/m^3 gold, a deposit of 30 x 10^6 m^3 would yield 450,000 oz of gold. At a price of $400 US/oz, the expected gross revenue would be $180 million. A dredge operation of 13,000 m^3/day could recover the deposit in 10 years, assuming an approximate 6.5 months/year working time, owing to the East coast's unfavorable sea and weather conditions from late fall to late spring.

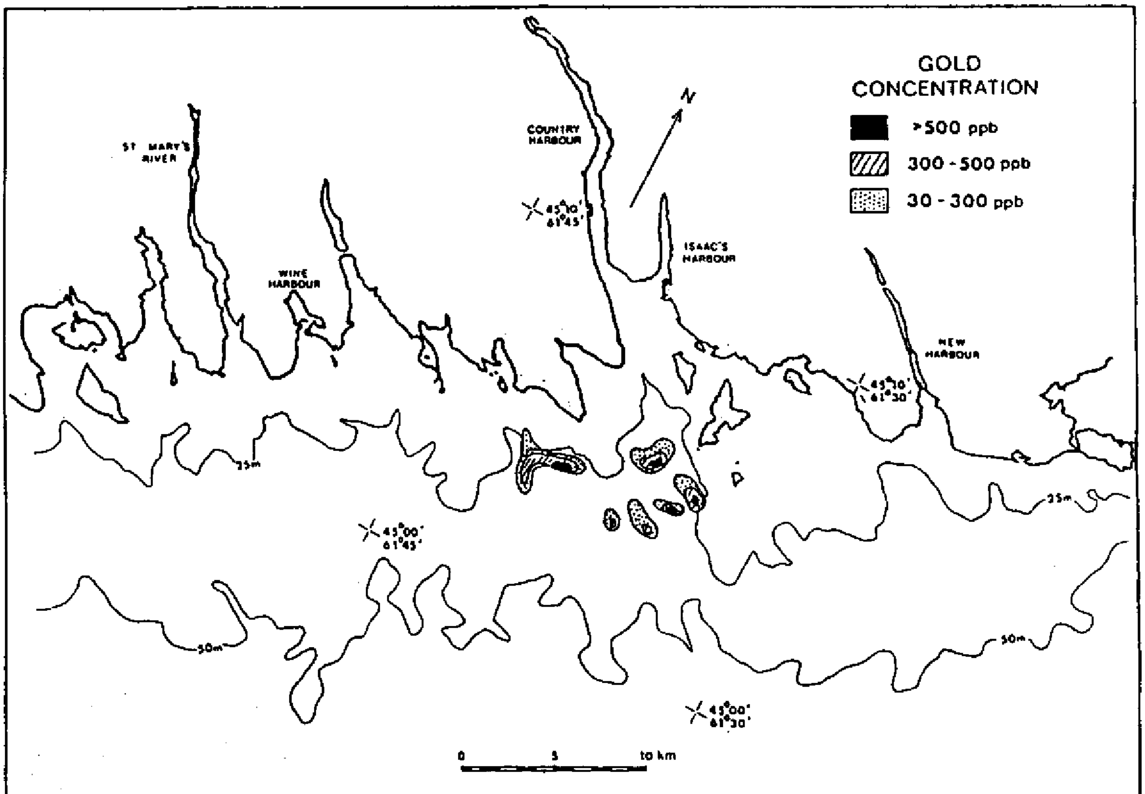

Figure 5. Gold concentration in the -63 micron sediment fraction (Hopkins, 1985)

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Biodata

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Baltimore Harbor Channels: Responding to the Increase in Ship Drafts

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Abstract

This paper traces the development of the Baltimore Harbor Channels in response to the increase in ship draft from colonial times to present. Emphasis is placed on the evolution of ship draft and the types of dredging equipment employed to meet the needs of the port of Baltimore. Her standing as a port has always been dramatically tied to dredging.

Baltimore was the cradle of dredging development. Out of economic necessity, she has had to dredge since colonial times. This early period saw the experimentation of several dredge types.

Dredging began under local funding, but evolved into a close partnership between city and federal government starting in the 1830s and peaking in the 1870s, when both governments had their own dredges working side by side.

The draft of 18th-century ocean-going vessels was generally less than 12 feet. These ships, small by today's standards, had no trouble navigating the coastal and estuarine waters of this country. Water depth first became a problem for these ships at the docks where ballast and refuse from the ships, as well as dirt and garbage washed down the streets and into the slips by rain runoff, gradually filled them in.

Today's Inner Harbor, the Northwest Branch of the Patapsco River, was the harbor for the port of Baltimore during colonial times and was referred to as the "Basin." It was a cove immediately upstream from Jones Falls. Like all colonial ports during the mid to late 1700s, Baltimore began experiencing the vexing problem of slips filling in. Because this problem was more from acts of man rather than acts of nature, the General Assembly in 1753 decreed that:

> ...no earth, sand or dirt was to be thrown into or put upon the beach or shore of Patapsco River, or any navigable branch thereof below high-water mark, unless secured by stone walls, dove-tailed log pens, &c., from washing into the river, under a penalty of five pounds.

However, neither man nor nature totally complied with the law and the problem continued.

To combat the continuing problem of harbor and slip sedimentation, the General Assembly appointed port wardens in 1783 and directed them to remove obstructions to navigation. To defray these expenses, all vessels entering and clearing the port were taxed at the rate of one penny per ton.

Baltimore acquired a dredge sometime around 1791. It was constructed locally but records don't say whether it was a man- or horse-operated dipper dredge. This dredge, or "mud machine" as the port wardens called it, spent most of its efforts removing the refuse and silt that built up alongside the docks. As the years went by, the channel in the Basin began to require a greater amount of attention. Dirt washing down the streets into the Basin and sediment from Jones Falls were creating shoals interfering with navigation.

By 1800 the depth of the channel had decreased to 6 feet in some locations. As a result the mayor resolved to dredge a channel 200 feet wide and 12 feet deep. The cost of dredging that year was $2,274.63. The dredged spoil was barged to various wharves where it was scooped out and used to fill in wharves as well as low ground. Property owners paid the city five shillings per scow load (300 cubic feet). Progress was being made. However, the task was more than the dredge could accomplish—it wore out in the attempt. By the time the dredge was replaced, much of the channel improvement was nullified.

Now the mayor was content to strive for a 10-foot channel. The struggle to maintain a navigable channel resumed in 1802. For the next 13 years the city continued to operate with only one dredge which, at best, was able to maintain a 10-foot-deep channel. Fortunately, for the port ship drafts had not increased from the previous century.

Around 1815, despite the increased capacity of successive dredges, Baltimore was forced to increase its dredge fleet to three horse-operated machines. However, the rate of shoaling had also increased, and even the three dredges combined could barely maintain a channel 11 feet deep.

In 1825 the port wardens acknowledged that the city's three dredges could not keep pace with the harbor's shoaling rate. Taking advantage of the problem, two residents who operated a foundry, John Watchman and John Bratt, proposed building a steam-powered dredge for the city. They claimed that their invention would be able to dredge 1500 tons of sediment a day compared to the 300 tons per day being removed by the city's fleet.

The city commissioners and port wardens reviewed Watchman and Bratt's model and surveyed ship captains who had seen steam dredges in operation in Europe and concluded that the proposal was sound. For some reason the city council did not act on the matter for another year. Finally, in April of 1826 they placed advertisements in Baltimore, Philadelphia and New York newspapers requesting bids for a steam-powered dredge rated at 12 horsepower. Watchman and Bratt won the contract but their dredge was not completed until late in 1827. Meanwhile, the channel had degraded to 10 feet in depth and groundings of the larger-class vessels became increasingly more frequent.

The port wardens were favorably impressed by Watchman and Bratt's product. It was a ladder bucket dredge operated by a 12-horsepower, low-pressure steam engine. The machinery was mounted on a paddlewheel steamboat — another innovation specified in the contract.

Such power for dredging had never been seen before in Baltimore. As each bucket on the "endless" chain (like a conveyer belt) would hit against the river bed, it would force the dredge backwards. The city contracted with Watchman and Bratt to remedy the problem with appropriate anchors and associated "moving apparatus." Once this was accomplished, the fortunes of the harbor took an upturn.

At last Baltimore had a dredge of sufficient capacity to begin to deepen the channel. Because the production of the dredge was so great in comparison to the city's other dredges, it had to be attended by four scows and a tow boat to carry off the dredged spoils. The higher dredging rate outpaced the demand for fill material and an out-of-the-way open water disposal area had to be found. The extra distance required to transport the dredged material translated into the need for a fifth scow. However, this problem was far outweighed by the prospects of channel improvement.

Other improvements to the steam dredge became necessary: the hull needed copper sheathing, the buckets were too large for the engine and were replaced with smaller ones equipped with iron teeth, and a more efficient copper boiler replaced the iron boiler. All of these expenses were still worth the price. However, Watchman and Bratt's dredge only removed 300 tons per day — far less than the original claim but still equal to the combined effort of the rest of the fleet. And what was more important was that by 1833, ships drawing 12 feet were now able to enter the harbor. To produce these results, 108,755 tons of dredged spoil were removed from the channel that year. The next year the channel was deepened further allowing ships drawing 13 feet to call and clear. This was sufficient for the size of most of the ships of the day.

Despite wharfage fees, tonnage taxes and even the renting out of the horse-operated dredges, the expense of dredging was a major public burden. For the period of 1798 to 1830, Baltimore spent over $400,000 on channel maintenance. In 1829 the city petitioned Congress for funding. It was not until 1836 that the federal government finally began the task of maintaining the Baltimore Harbor Channels.

In 1836, after several years of lobbying, Congress appropriated $20,000 for channel deepening. In a unique move, the funds were given to Baltimore and the dredging was performed under the direction of the port wardens. Another $35,000 was appropriated in 1838, but these were the last federal monies until 1852 when $20,000 was again appropriated. This appropriation was matched by the city and a new era of dredging was begun.

In 1850 Baltimore was the third largest city but was losing port traffic due to channel limitations. The limiting depths of the natural channel from the Chesapeake Bay up the Patapsco River varied from 16 to 18 feet, but in 1850 over 15 percent of the ships drew 18 feet of water or more. Captain Henry Brewerton, United States Corps of Engineers, was placed in charge of channel improvements in 1852. He united with the Board of Commissioners of the Patapsco River Improvement to formulate a plan of action. After consultation with the Board of Commissioners and ship pilots, he recommended a channel 150 feet wide and 22 feet deep at mean low water, extending in a direct line from Fort McHenry to a point one-and-one-half miles below Fort Carroll, and thence in another straight distance of nine miles out into the Chesapeake Bay. This became known as Brewerton's Channel.

After examination of the city of Baltimore's lone dredge, Brewerton decided it was inadequate for channel dredging. He conferred with fellow Corps officers and concluded that a dipper dredge would be best suited for channel work. He was particularly impressed with the Osgood patented dredge being built by A.B. Cooley in Philadelphia. Upon his recommendation, the city contracted for the building of the first in December of 1852, and he contracted for a second the following April.

In October 1853, operations were commenced under Captain Brewerton's supervision with a force of two dredges. Because the upper reaches of the Patapsco (between forts McHenry and Carroll) were from 19 to 21 feet deep, the initial dredging was performed in the lower nine-mile reach, where natural depths over the shoals varied from 16 to 18 feet. Dredging was suspended on December 17, 1853, and resumed the following May. The city added another dredge in July, and the three worked until December 2, 1854. Dredging continued in that pattern annually with the Corps of Engineers adding two more dredges in May 1857. The Corps purchased a fourth dredge in March 1858.

Federal appropriations were exhausted in September 1858. However, the city continued funding the operation using one Corps dredge and their two until 1860. A hydrographic survey conducted in November of 1859 revealed that six of the nine miles had been dredged to an average of 23 1/2 feet.

In 1860 the four government dredges, Patapsco, Susquehanna, Chesapeake and Potomac, were loaned to private dock owners. During the Civil War they were loaned out to various military departments.

After the war the harbor development shifted back to the Corps of Engineers under the direction of William P. Craighill. A hydrographic survey of the dredged channel in 1866 revealed that the eastern extremity had shoaled considerably but not the more interior reaches. Therefore, a new entrance channel was laid out more in line with the Susquehanna currents. The new channel was to 200 feet wide and was assigned the name of the new Corps officer in charge — Craighill.

Major Craighill repaired three of the former government dredges and placed them back in service. In 1869 the Craighill channel was completed to a width of 200 feet and a depth of 21 feet, and the Brewerton Channel had been restored to the same depth but an unspecified limited width (probably the original 150 feet).

However, both the width and depth were inadequate for the larger ships of the period. In 1866 a 21-foot channel could accommodate all but about 97 percent of the voyages but by 1870, 15 percent of the vessels were capable of drawing over 21 feet of water. As a consequence, Baltimore was handling a smaller percentage of tonnage compared to New York and Philadelphia than previously. The latter two ports prospered in part from their natural advantage of tide range and channel depth.

Maintenance dredging resumed in 1870 after the three government dredges had been thoroughly repaired. However, by the end of the dredging season it was obvious that the dredges were outdated and not worth rehabilitating, so they were sold. A $50,000 appropriation for 1871 afforded maintenance dredging under contract.

The city was desperate to regain her national prominence as a port, and in 1872 raised $200,000 for channel deepening. Congress responded with an additional $100,000 and plans were made to deepen the harbor to 24 feet with widths varying from 250 to 400 feet. The city and the Corps let contracts for the removal of 450,000 and 300,000 cubic yards, respectively.

By the summer of 1872 the largest gathering of dredges in the U.S. — thirteen — were busy enlarging the channels. Even with this armada, it took until the spring of 1874 to remove the additional 2 million cubic yards necessary to complete the project.

Both clamshell and dipper dredges were used. Craighill was greatly impressed with the clamshells, which averaged over 90 cubic yards per hour and sometimes reached as high as 300 yards. This compared to the former government dipper dredges, which averaged only 25 yards per hour. Ironically, a Baltimorean had patented a cutter suction dredge in 1867, and even though the Corps was having success using a hydraulic dredge along the southern Atlantic as early as 1872, it was not considered for the Baltimore project.

In 1881 Congress approved and funded deepening of the channel to 27 feet. The channel deepening was completed in 1884, bringing a boost to port trade. By 1889 the port of Baltimore was handling over 3.2 million tons of traffic in a channel whose depths were now comparable to those of Philadelphia and New York. Over the next three years the channel was widened to 600 feet throughout by the National and American Dredging companies. Despite these improvements, New York was still the largest port.

The 30-foot channel was approved in 1896 and estimated to take six years to complete at a cost of $2 million. However, by 1901 Baltimore was pushing for a channel 35 feet deep and 1000 feet wide. This was necessary to keep up with the rapidly increasing ship drafts. The maximum drafts in 1890 were 27 1/2 feet but by 1895 they had increased to 29 feet and by 1900 to 32 1/2 feet! The increased drafts required greater underkeel clearance because of trim and ship motion problems associated with longer ships.

In 1905 Congress approved dredging the approach channels to 35 feet but holding the channel width to 600 feet. By the time this project was completed in 1915, ship drafts had exceeded 35 feet. The Corps did not contract for dredging of the inner harbor until 1916. From that time until 1954, Baltimore's limiting channel depths ranged from 35 to 37 feet despite the fact that ship drafts continued to increase.

Ship drafts reached 40 feet by 1930 but not until 1954 did the Baltimore channels reach 39 feet. Six years later Congress appropriated $1.9 million to dredge the channels to 42 feet. In 1970 Congress authorized a 50-foot channel.

The original port area of Baltimore was built around the inner harbor. During colonial times the natural depths of 15 to 20 feet were more than sufficient for ships of the time. The inner harbor quickly began to shoal such that the channels degraded to 10 feet during the federal period. First city-owned and contracted hand- and horse-operated dipper dredges, followed by steam-powered ladder bucket dredges, were used to restore the channels to their natural depths. About the time this was achieved, ship drafts started increasing to the point where the natural channel of the Patapsco River became inadequate.

The Corps began dredging in 1852, resorting back to dipper dredges. The clamshell dredge was introduced in the 1870s and continued to work alongside the dipper well into the 1900s. It was not until recent decades that the hydraulic dredge was used in Baltimore, and that despite the fact that Corps-owned hydraulic dredges were built in Baltimore for other Corps projects before 1911. While most major ports were deepened by Corps dredges, from 1870 to 1930, dredging in Baltimore was performed by contractors.

From colonial times to the present, Baltimore has had to resort to dredging to keep pace with the evolution of increasing ship sizes. When the 50-foot channel is completed, Baltimore will once again be competitive with rival Atlantic coast ports.

Biodata

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Dave Bastian is a civil engineer with the Navigation Analysis Center at the Corps of Engineers' Institute for Water Resources at Fort Belvoir. Prior to his present position, he had been in charge of the Chesapeake Bay Model Project — the world's largest physical hydraulic model. As a matter of fact, he had made a presentation to a previous Texas A&M Dredging Seminar about the effects of dredging the 50-foot Baltimore Harbor Channels on Chesapeake Bay salinities. This was based upon his work at the model. Our previous speaker, Ginny Pankow, was one of his employees there.

Dave received degrees from Georgia Tech and Delft, where he specialized in river engineering. It was during his studies at Delft that he became interested in historic dredging. This interest has developed into a major hobby resulting in numerous presentations and publications dealing with the history of dredging.

Criteria for Subaqueous Borrow-Pit Disposal Sites

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Introduction

In 1977, the New York District of the U.S. Army Corps of Engineers began a comprehensive study of alternative methods of dredged sediment disposal (Conner et al., 1979). The burial of dredged sediment in subaqueous borrow pits was one of three alternatives that were deemed possible in special cases and feasible for large volumes of dredged materials (the other two options were shallow ocean disposal and confined upland disposal). A variety of studies have been completed not only to examine the particulars of such an operation in New York Harbor (e.g. Bokuniewicz, 1983) but also to investigate generic processes that would be involved in the implementation of this disposal alternative. These include the studies of covering or capping dredged sediment with sand at subaqueous sites (Morton, 1983; O'Connor, 1982); the consolidation of capped deposits (Demars et al., 1984); the stability of sand caps (Freeland et al., 1983) and the isolation of contaminants by caps (O'Connor, 1982; Brannon et al., 1984). In addition, the burial and capping of contaminated dredged material in a subaqueous depression in the Duwanish Waterway, Seattle, Washington has been successfully completed (Sumeri, 1984). The basic principles of all the essential features of a borrow-pit disposal project have been demonstrated and an Environmental Impact Statement is being prepared to implement this disposal alternative in New York Harbor.

Some general technical criteria are discussed in this article for the selection or construction of borrow-pit disposal sites with emphasis on the New York metropolitan area.

Background

The size and shape of a suitable subaqueous pit for the burial of dredged sediment depends both on the physical limitations of the equipment and on the fate of dredged sediment released at open-water disposal sites. A great deal has been learned about the discharge process over the past decade and before I proceed to calculate the critical characteristics of pit disposal sites, it will be necessary to review some of the results and evidence upon which the calculation is made.

The disposal operation will be assumed to have the following characteristics:

(1) The dredged sediment will be fine-grained. Subaqueous burial is intended to be a disposal option for contaminated sediments and many of the most troublesome contaminants are associated with fine-grained sediments including petroleum hydrocarbons, heavy metals, polychlorinated biphenyls, and other chlorinated hydrocarbons, polynuclear aromatics, pesticides, and some radio-nuclides. As a result it is likely that the dredged sediment designated for burial will be fine-grained.

(2) The sediment will be dredged with a clamshell dredge and discharged from a barge. In addition to this being the most common method of dredging and disposal in the New York metropolitan region, there are some technical advantages to using this method. This method is most likely to result in the discharge of blocks of dredged sediment which will form a compact deposit on the sea floor (Bokuniewicz and Gordon, 1980). There are also limits to the strength of the dredged sediment deposit required to support the sand cap (Bokuniewicz and Liu, 1981) and blocks of sediment resulting from a clamshell dredging operation are most likely to retain a sufficient strength during the dredging and disposal process. In this region all deposits of dredged mud that have been successfully capped have been dredged with a clamshell dredge and discharged from a barge (e.g. Morton, 1983; O'Connor, 1982).

(3) The barges will be about 200 feet (61 m) long, 50 feet (15 m) wide, and draw 18 feet (5.5 m) of water. This is slightly larger than barges used, for example, by the American Dredge Company.

(4) Discharges will take place in water less than 220 feet (67 m) deep. This is the maximum water depth for which the discharge processes that will be next described have been observed (Bokuniewicz et al., 1978).

Such a disposal operation will have the following characteristics:

(1) During the disposal operation from a scow or hopper dredge, less than 5 percent of the released sediment will remian in suspension and to be dispersed from the disposal site. This conclusion was first reached by Gordon (1974). He made measurements during disposal operations in Long Island Sound and

showed that less than 1 percent of the dredged silt released at the disposal site remained in suspension long enough to be dispersed by the tides. A similar conclusion was reached by Sustar and Wakeman (1977) as a result of operations they made in San Francisco Bay. They found that only between 1 and 5 percent of the mud that was discharged remained in suspension above 2 m (6.6 feet) of the bottom. They also conducted laboratory experiements that reinforced their conclusion that the disposal operation causes very little disturbance in the upper part of the water column. A similar conclusion was reached by Bokuniewicz et al. (1978) from observations in Puget Sound, Long Island Sound, Lake Erie, and Lake Ontario.

At the Mud Dump Site on the Atlantic continental shelf outside of New York Harbor, a detailed accounting of the dry mass in the subsequently formed deposit at the disposal site showed that an average of about 4 percent was lost during transport and discharge (Tavolaro, 1984).

The same conclusion was reached after the study of three disposal mounds in Long Island Sound (Morton, 1983). The volume of dredged material on disposal sites was measured by careful bathymetric surveys and compared to the volume dredged, although the volume dredged was estimated by the volume in the scows and the uncertainty is relatively large (Morton, 1983). In each of two mounds, 95 percent of the amount discharged was found on the disposal site (Morton, 1983), indicating a loss of 5 percent. At the third site, 90 percent was found at the site but additional material was present beyond the immediate mound and "it was possible for significant amounts of dredged material to be undetected by acoustic measurements" (Morton, 1983).

The Mitre Report (Conner et al., 1979) also claims that almost all of the released sand and silt will be deposited quickly based on exploratory calculations for the New York Bight using the Tetra Tech model (Holiday et al., 1978; Brandsma and Divoky, 1976). In the model calculations, all of the sand and silt were deposited within about 20 minutes and within 200 yards (183 m) of the point of discharge. For a clay slurry the time may be considerably longer; some of the model calculations showed that three hours would be needed to deposit 90 percent of the clay particles that were released as a slurry from the scow or hopper. Of course, blocks of dredged mud would reach their terminal fall velocity quickly after discharge (Bokuniewicz and Gordon, 1980) and reach the bottom, presumably with little or no dispersion during descent.

(2) More than 95 percent of the sediment will be deposited on a flat sea floor within a few hundred yards of the discharge point. Blocks of cohesive sediment may either disintegrate or deposit intact upon impact with the bottom. The size of the block as well as its strength and the hardness of the sea floor all play a role in its fate (Bokuniewicz and Gordon, 1980). Blocks of silt and clay smaller than 0.85 m (2.8 feet) in diameter are unlikely to fragment upon impact with a hard sea floor (Bokuniewicz and Gordon, 1980). Clods about 0.2 m (0.7 feet) in diameter were found on the surface of one disposal mound in Long Island Sound (Bokuniewicz and Gordon, 1980) and clods of cohesive sediment with diameters of about 0.4 m (1.3 feet) were found on another (Morton and Miller, 1980)

If the blocks do disintegrate upon impact, it is likely that the residue will join a slurry of dredged material and be incorporated into a thin, dense bottom surge (e.g. Proni, 1982) that contains almost all of the dredged sediment released except for that contained in the surviving blocks. Over a flat bottom, the sediment is deposited within a few hundred yards of the point of release. This has been documented under a wide range of conditions (Bokuniewicz et al., 1978). Discharges of muddy sediment from a hopper dredge in water 18 m deep in Lake Erie were monitored to show that the surge did not carry material farther than about 200 m (220 yards) from the impact point over a flat disposal area (Bokuniewicz et al., 1978). At this same site later, more than 70 percent of the dredge sediment was found within about 250 m (273 yards) of the designated discharge point (Danek et al., 1977); some of the missing material (an unspecified amount) was not found on the site because it had been released at another location. During a disposal operation in Long Island Sound, 80 percent of the 1.2 million cubic meters (1.6 million cubic yards) of muddy dredged sediment that was discharged from scows in about 20 m (66 feet) of water was deposited within 30 m (33 yards) of the center of the discharge location and 90 percent within 120 m (131 yards) (Gordon, 1974). At each of three sites in Long Island Sound studied by Morton (1983), 90 percent, 95 percent, and 95 percent of the material discharged was found within 200 m (220 yards) of each discharge point.

Direct observations of spread of bottom surges were also made in a borrow pit in New York Harbor (Bokuniewicz, 1985). The disposal operation was done with barges. Although much of the dredged sediment was released as cohesive blocks, there was enough fluid sediment to produce bottom surges like those described by Bokuniewicz et al., 1978. Forty discharges were monitored. Surges were detected on 33 of these. Only once was a surge detected farther than 110 m (121 yards) from the discharge point; that one was seen at a distance of about 175 m (193 yards). The surge was not detected seven times at distances between 70 m and 110 m (77 yards and 121 yards) from the discharge point.

(3) Compact, quasi-conical deposits can be built by repeated discharges at the same location. The shape of deposits during open-water disposal operations can be forecast in light of available observations. The diameter of potential deposits is limited by the range of the bottom surge that is formed during the disposal operation and very compact deposits can be created by point-dumping (e.g. Bokuniewicz and Gordon, 1980; Morton, 1983). The side slopes of the deposit depend primarily on the character of the material. In principle, clods and coarse sediment could accumulate on the disposal site in a pile with side slope reaching the angle of repose for coarse material, 35 degrees. Clods were found on the surface of a disposal mound in Long Island Sound which had been formed by open-water disposal of muddy sediment (Bokuniewicz and Gordon, 1980). The deposit had an average slope of 6 degrees near its peak although locally steeper slopes were seen (Bokuniewicz and Gordon, 1980). Two other deposits also have been created near this same site (Morton, 1983). The larger contains 118,000 cubic meters (154,344 cubic yards) of mud. It has a radius of about 100 m (110 yards) and side slopes as steep as 7 degrees; clods of cohesive sediment also were found on its surface (Morton and Miller, 1980). The smaller deposit consisted of a mound of mud, which contained 26,000 cubic meters (34,008 cubic yards) and had a radius of 100 m (110 yards) and side slopes as steep as 6 degrees, covered with a layer of sand. The combined deposit contained 60,000 cubic meters (78,480 cubic yards). Its radius was about 200 m (220 yards) and the side slopes reached angles as high as 8 degrees. During a discharge operation in Puget Sound, clods were detected leaving the scow and the resulting deposit here had slopes as steep as 2 or 3 degrees (Bokuniewicz et al., 1978). At a disposal site on the Atlantic shelf off the mouth of Chesapeake Bay, about 650,000 cubic meters (850,200 cubic yards) of loose silt and very fine sand was discharged to created mounds 3.3 m high (11 feet) with average sideslopes of about 2 degrees (Hands and DeLoach, 1984).

Deposition of fine-grained sediment from a bottom surge produces a dredged sediment deposit with low side slope. Observations of surges in Lake Erie have been used to calculate the maximum slopes for deposits formed in this way (Bokuniewicz and Gordon, 1980). The maximum slope is the slope at which the energy gained by the surge as it runs down the slope is equal to the empirical rate of energy dissipation calculated from observations of spreading surges (Bokuniewicz et al., 1978). At the maximum slope, the surge should travel indefinitely without losing energy and, presumably, without depositing its sediment. The maximum slope has been calculated to be about 3 degrees (Bokuniewicz and Gordon, 1980; Bokuniewicz, 1983). Such low slopes were found on the flanks of a deposit of dredged mud in Long Island Sound (Bokuniewicz and Gordon, 1980). A dredged sediment deposit in Chesapeake Bay was found to have a maximum surface slope of about 0.59 degrees and an average slope of 0.12 degrees (Biggs, 1970). After a disposal operation in Lake Erie, the maximum slope of the deposit's surface was 0.3 degrees (Alther and Wyeth, 1980). During laboratory tank tests to simulate open-water disposals of dredged mud, mounds were formed with slopes on the order of 0.3 degrees (Chase, undated). In all of these cases, it appeared that the sediment had been deposited from a slurry.

The number of studies is relatively small and there is not yet a generalized model that is widely accepted and available to describe all the relevant processes and to predict the form of the deposit. Nevertheless, the available studies may be used as a basis for forecasting the form of deposits of dredged sediment if we assume that point-dumping will be done in relatively shallow (20 m, 66 feet) water. Enough information is at hand to consider four classes of material — cohesive mud, fluid mud, sand, and a mixture of sand and fluid mud. The cohesive mud is likely to have been dredged with a clamshell-bucket dredge and the deposit formed primarily of clods of material. In this case we expect to find a deposit with slopes of less than 30 degrees, but experience has shown that the slopes will probably be 2 degrees to 8 degrees. The central mound of clods will be surrounded by a blanket composed of fine-grained material that had been deposited from a bottom surge formed by ablation of clods, entrainment of water during descent, and the disintegration of some clods upon impact. The surface slopes of the apron should be less than 3 degrees and experience has shown that they will probably be less than 1 degree. An example of such a deposit was formed in Long Island Sound (Bokuniewicz and Gordon, 1980).

Fine-grained sediment dredged hydraulically will most likely be a very weak and fluid sediment in the hoppers or a very dense slurry. The expected bulk specific gravity of such material would be between 1.1 and 1.3 (Bokuniewicz, 1979). This material will produce a deposit with a minimum radius of about 200 m (220 yards) and side slopes of less than 3 degrees. Experience has shown that actual side slopes will probably be less than 1 degree. An example of such a deposit was created in Lake Erie (Alther and Wyeth, 1980).

There is less experience to draw on to make a forecast for the form of a sandy deposit. If we assume that the sand is sufficiently coarse not to be carried out of the impact area by a bottom surge, then a deposit with side slopes less than about 30 degrees and probably less than about 3 degrees will be created. An example of such a deposit was described by Morton (1983). A mixture of dredged sand and mud is likely to segregate during the disposal operation. In this case we might expect to find a deposit with a central mound of coarse-grained material having side slopes of about 8 degrees surrounded by an apron of fine-grained sediment with side slopes of about 1 degree, similar in shape to that formed by the discharge of cohesive mud.

This information was used to predict the implacement of a submerged sand ridge in New York Harbor (Bokuniewicz, 1982). The ridge was constructed in December 1981 by the hopper dredge *Goethals* using sand from Ambrose Channel. The deposit that was created by the *Goethals* was in a form that was very close to the predicted form (Bokuniewicz, 1982). The average water depth over the ridge crest was 39 feet (11.9 m); the predicted value was 37 feet (11.3 m). The 50-foot contour was displaced about 270 yards (247 m) to the north as predicted and the location of the lowest points along the ridge crest were to the east and west of the center as predicted. The predicted side slopes were about 1.6 degrees and the actual slopes were later found to average 1.0 degrees.

(4) At the disposal site, bottom bathymetry with slopes of a few degrees or more will substantially limit the spread of dredged material during the discharge process. There are two lines of evidence for this conclusion. The first is an empirical calculation based on observations of the behavior of the spread of dredged sediment over a flat disposal site floor. The other is the direct observation of the effects of slopes on the spread of material.

If the bottom surge must run up a slope, the distance it can travel must be less than it could travel over a flat sea floor. In the barge, the dredged sediment is characterized by a specific amount of potential energy. During the discharge process, potential energy is converted to kinetic energy and dissipated through friction. When all its initial energy has been dissipated, the sediment comes to rest on the sea floor. All other things being equal, a surge that is travelling up a slope uses up energy more quickly than one running over a flat sea floor since, in addition to all the frictional mechanisms of dissipation, work must also be done to raise its center of mass continually. As a result, it depletes its energy more quickly and comes to rest sooner before it can travel as far. Investigators in the Duwanish River concluded that "relatively shallow depressions with steep side slopes appear to significantly reduce the outward surge of dumped cohesive dredged material" (Sumeri, 1984).

For the design of a disposal operation, the losses of energy must be quantified. This has been done for a unique and extensive set of data collected under the U.S. Army Corps of Engineers Dredged Material Research Program. Observations of about 30 discharges of muddy sediment from a hopper dredge were made with current meters, tranmissometers, pumped water samples and echo sounders. The details of this study are given by Bokuniewicz (1985). In the former report the size, shape, position, mass, and energy of the bottom surge were presented at various times after discharge. Some of this data is shown in Figure 1. In this figure the dots show the total energy of the surge as it moves away from the discharge point along a flat disposal site floor. The line labeled H in Figure 1 helps to show the general trend of decreasing energy as the surge moves outward. The amount of energy used in rising a unit volume of the surge a height h is $(\rho - \rho_o)$ g h where ρ is the bulk density, ρ_o is the density of water and g is the acceleration of gravity. If the surge is climbing a slope of angle α, the additional energy required to cover a distance R is $(\rho - \rho_o)$ g R tan α. The curves superimposed on Figure 1 indicate the amount of work required to lift the surge up various slopes calculated in this way. These curves are not straight lines because the mass of the surge is decreasing, as well as its energy, as it moves outward. Where the curved lines cross the line "H," the energy in the surge spreading horizontally is just equal to that needed to climb the indicated slope at the specified distance from the discharge point. If the surge had been climbing such a slope the additional energy requirement would have required all its energy at that distance and the spread would have stopped. For example, the curve indicating the energy needed to climb a slope of 3 degrees crosses line "H" at a distance of about 65 m (72 yards) from the discharge point. If the surge had been climbing a slope of 3 degrees, all its energy would have been required to reach this point and it would spread no further. Actually, it would stop before it reached this point because a correction has not been made to account for a more rapid decrease of the surge's mass as it slows more quickly moving up the slope. Nevertheless, the calculations show that even low slopes can substantially limit the spread of the surge; a slope of 3 degrees in the example reduces the distance the surge can travel to 72 yards (66 m) or by about 30 percent of its run over a flat disposal site floor.

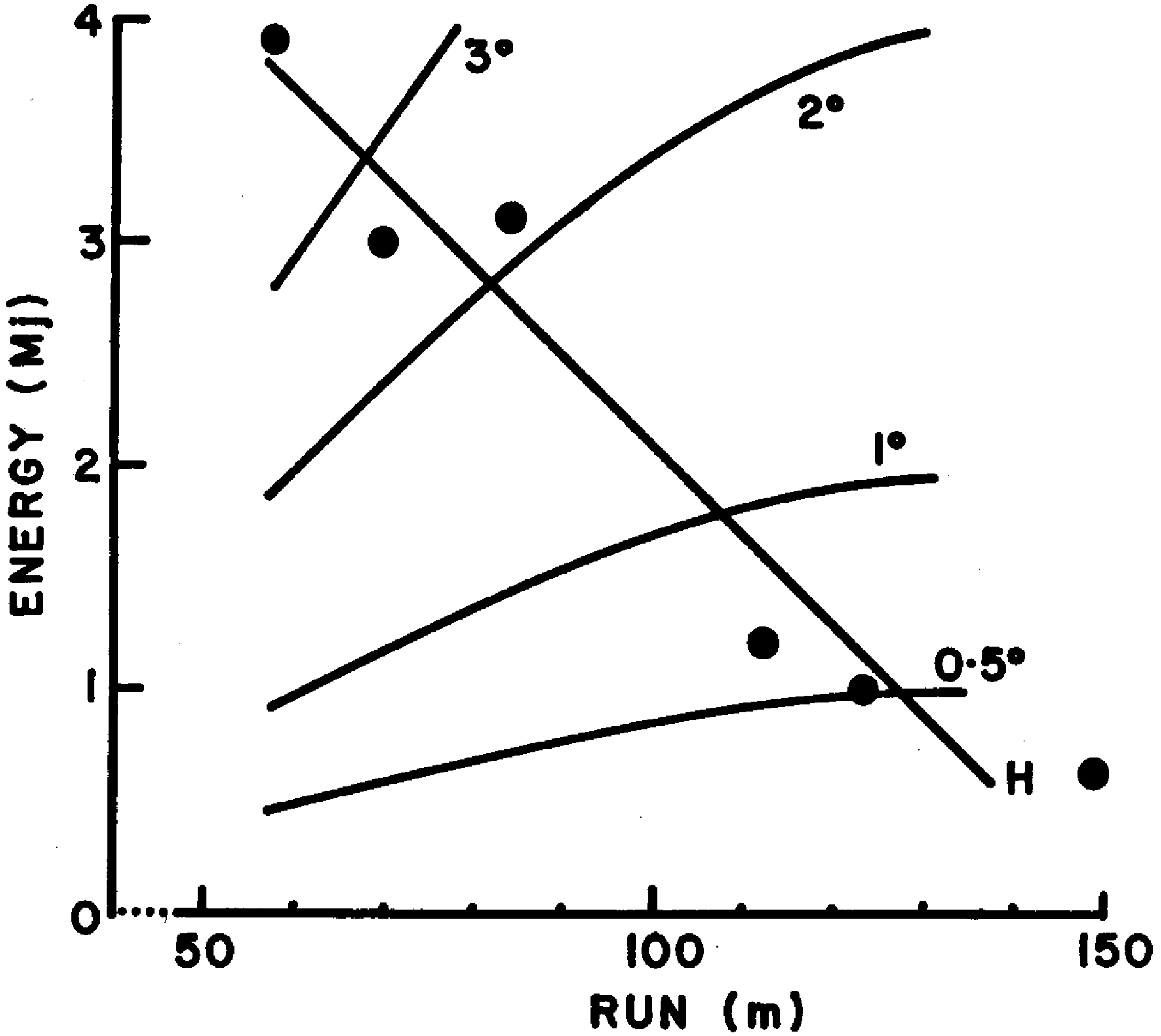

Figure 1. Total energy H in the surge measured as a function of the position of the surge front. The curved lines (labeled 0.5°, 1°, 2° and 3°) represent the work required to move the surge up the indicated slope. The intersection of a curve with H marks the maximum travel of the surge up that slope.

Regardless of the slope, at any point in the travel of the surge, we can calculate how high it would have to rise in order to come to rest. For example, soon after impact on the disposal site the surge had an energy density of 150 Joules/m³ and a bulk specific gravity of 1.004 (Bokuniewicz et al., 1978). If I ignore all other forms of energy dissipation, such as frictional losses, I find that raising a unit volume a height of 4.3 m (14.2 feet) would require all of its 150 Joules. Regardless of the slope, the surge could not rise higher than 4.3 m (14.2 feet). When the head of the surge had reached a distance of 112 m (123 yards) from the point of impact, much of its energy had been dissipated. Its energy density here was about 20 Joules/m³ and its effective specific gravity was about 1.0015. At this location, a rise of 1.3 m (4.3 feet) would bring the surge to rest. This prediction has been supported by observations of a disposal operation near the Duwamish Waterway in Seattle, Washington (Sumeri, 1984). During this operation, silt was dredged with a clamshell dredge and discharged in 20 m (66 feet) of water over a depression in the river floor that measured about 30 m (33 yards) wide, 140 m (153 yards) long and up to 2.4 m (8 feet) deep below the ambient sea floor. The side slopes of this depression were as steep as 11 to 20 degrees. Even though the depression was relatively shallow, the side slopes significantly reduced the outward surge of the discharged material so that nearly all of the released sediment was deposited in the depression (Sumeri, 1984).

During the disposal operation in New York Harbor that was described earlier, two discharges were monitored at the wall of a pit (Bokuniewicz, 1985). During the first, the barge was 83 m from the rim of the pit and the pit floor under the scow was about 7 m lower than the ambient sea floor. The wall slope was about 5 degrees. About 2 minutes after the dredged sediment was released, a surge 3 m (10 feet) thick was seen on the fathometer record at a distance of 18 m (20 feet) upslope from the discharging point. This surge was moving relatively slowly up the slope. A second observation boat over the rim of the pit did not detect the surge; the surge did not escape from the pit as might have been anticipated from the preceding energy calculations.

In the second discharge that was monitored at the pit wall, the barge was 55 m (60 yards) from the rim of the pit and the pit floor under the barge was 7 m (23 feet) lower than the ambient sea floor. The slope here was about 9 degrees. A surge 6.1 m (20 feet) thick was detected 43 seconds after the discharge at a distance of 11 m from the discharge point. A second observation boat was 11 m (36 feet) farther upslope and detected the surge 13 seconds later. The surge was spreading at a rate of 0.9 m/s (3 feet/s) upslope. At this time, I have estimated that the energy density of the surge was about 480 J/m ^{3}. This is higher than the initial energy density calculated by Bokuniewicz et al. (1978), probably because the measurements in New York Harbor were made relatively close to the discharge point; the surge had not spread far and, as a result, the surge energy was still concentrated in a relatively small volume. The subsequent spread of these surges, however, was the same as that observed by Bokuniewicz et al. (1978), so after an initial rapid dilution of energy, the energy densities, and the energy dissipation rates, were likely the same. The surge arrived at the rim of the pit about 3 minutes after discharge. In climbing the pit wall, it had raised its center of mass about 4.9 m (16 feet) with an estimated total energy demand of 430 J/m^3. It appears, therefore, that the surge had spent nearly all of its energy in climbing the slope.

Size of the Disposal Site

The minimum conditions for an acceptable pit may now be specified based on the following conditions:

(1) Clods of dredged material will be deposited at the discharge point and the bottom surge generated will not spread more than 220 yards (200 m) from the point of impact. This is its limit over a flat disposal site and the presence of sloping walls will limit its spread even farther.

(2) The initial energy density of the bottom surge at the discharge point will be about 500 Joules/m^3. This is slightly more than the highest estimated value based on the observations.

(3) Subsequent to impact, the energy levels and dissipation rates will be approximately as described by Bokuniewicz et al., 1978. This is the only available data and the spread of the surges observed in New York Harbor and the sizes of deposits at other locations are consistent with this empirical model.

With these results, a range of pit radii, wall slopes and depths can be specified to prevent the escape of the bottom surge of dredged sediment from the pit. For example:

(1) If the side slopes are less than 1 degree, the radius must be at least 220 yards (200 m).

(2) If the side slopes are greater than 1 degree, (a) the pit floor must be at least 5 feet (1.5 m) below the ambient sea floor and (b) the pit must be wide enough so that discharges can occur at least 123 yards (113 m) from the bathymetric contour that is 5 feet (1.5 m) below the ambient sea floor. For side slopes of 1 degree within 5 feet (1.5 m) of the ambient sea floor, this gives the pit a radius of 220 yards (200 m). The radius will be less for steeper slopes but since the angle of repose of sand is about 30 degrees, it will never be less than about 125 yards (114 m) unless the pit is deeper.

(3) Pits of minimum radius must have floors at least 45 feet (13.7 m) below the ambient sea floor. (This is the rise needed to absorb 500 Joules/m^3.) For side slopes of 7 degrees the minimum radius would be about 123 yards (113 m). At the angle of repose for sand, in principle the radius would only need to be 26 yards (24 m) but such small pits obviously are beyond the physical limits of the equipment to be used.

It is my opinion that the most useful practical criterion is that the pit floor must be deeper than 5 feet (1.5 m) below the ambient sea floor everywhere within 123 yards (113 m) of the discharge point. The intended discharge point can be marked with a taut-wire buoy with a watch circle radius of about 5 percent of the water depth but the usual marker will have a watch circle about equal to twice the water depth. In a carefully controlled operation the actual discharge point may be expected to be within one and a half barge lengths of the intended point or within about 100 yards (91 m) from the buoy. If we assume that the ambient water depth is sufficient for the barges to reach the pit from any direction, the pit must be deeper than 23 feet (7 m) over an area with a radius of about 250 yards (229 m). The side slopes should be as steep as possible outside of this area to minimize the area covered by the pit. For sand the maximum slope is about 30 degrees. In principle, there is no reason why the pit could not be created in mud. In digging the pit, however, sand may be useful for beach nourishment, construction fill, or aggregate. Either sand or mud could be used for capping material, but if mud was not used for capping, the excavated mud would have to be disposed at another site.

Erosion Potential

There is reasonable understanding of the sediment transport processes involving coarse-grained, noncohesive sediments and some predictive models have recently been developed, (e.g. Freeland, et al., 1983). Progress towards understanding the erosion, transportation, and deposition of fine-grained sediment has been much slower. As a result, there are no widely accepted and tested general models to predict the erosion of a mound of fine-grained dredged sediment by waves and currents. At the present time, we have very little reliable, predictive capability even though much work has been done in this field.

In this section, I will briefly state some of the reasons why this prediction is still difficult in light of recent research and then discuss the type of circumstantial evidence we could amass to estimate the vulnerability of a deposit to erosion even without a general predictive model.

Many of the problems with predicting sediment transport arise because there is not a linear relation between the currents and the movement of sediment. It is often difficult and costly to predict the currents in a specific region, but even if we knew what the currents were to a reasonable accuracy, our models of the transport of sediment would be subject to relatively large uncertainties. The rate of sediment transport, for example, is roughly proportional to the cube of the current velocity. As a result, a small or acceptable uncertainty in the measurement (or prediction) of the currents can make a disproportionately large uncertainty in the calculated rate of sediment transport. These sorts of problems, however, are not necessarily fatal and good progress has been made modelling coarse-grained noncohesive sediments despite this difficulty. Other problems plague the effort to model fine-grained, cohesive sediment transport.

First, there appears to be no single relationship between the physical properties of a cohesive sediment and the current velocity needed to initiate erosion (i.e., the critical erosion velocity). Neither has a general quantitative relationship between the activity of benthic animals and the critical erosion velocity even though many studies have shown the sensitivity of the erosion to benthic activity (e.g. Rhoads et al., 1978; Nowell et al., 1981).

The importance of the roughness of the sediment surface was dramatically realized during monitoring of disposal mounds in Long Island Sound (Morton and Miller, 1980). After the passage of a hurricane over the area, the top of one mound was truncated; a layer of sediment about 2 m (6.6 feet) thick (or about 9,900 cubic meters, 13,000 cubic yards) had been removed and the top surface of the mound, which had been rounded in profile with a minimum depth of 17 m (56 feet), was now flat at a depth of about 19 m (62 feet). Two other mounds of dredged sediment were in the near vicinity and had minimum depths of less than 19 m (62 feet) but neither of these two showed evidence of erosion. The difference in behavior between the mound that suffered erosion and these other two was explained through differences in their physical properties. The two mounds that survived unaltered by the hurricane had smooth, fine sand surfaces while the mound that was eroded had a rough surface characterized by clods of cohesive mud. Calculations were presented (Morton and Miller, 1980) to show that the high roughness resulting from the clods of sediment on one mound created a greater fluid shear stress and caused the sediment to erode under the combined effects of storm waves and currents while the smoother surface of the other mound resulted in smaller fluid stresses that were not capable of eroding the sediment surface. These investigators, however, point out that the calculation of fluid stresses under the combined effects of waves and currents are extremely complicated; that the mode of failure of cohesive clods under high shear stresses in unknown; and that the partitioning of shear stresses over rough beds under the combined action of waves and currents is likewise unknown.

In addition to the aforementioned difficulties with calculating the critical erosion velocities for fine-grained cohesive sediments, little is known about the rates in the laboratory on abiotic sediments; only a few of these were done in salt water (e.g., Mehta et al., 1982). Empirical formulas are available from these studies, but there is no widely accepted general form nor has there been field verification of these relationships. As I have mentioned earlier, some modelling of fine-sediment resuspension transport and deposition is being done by other investigators applying one or another of the empirical expressions for the resuspension rate, but these models must be considered experimental.

In light of these difficulties and uncertainties, mathematical models of sediment transport will be costly, time-consuming, and likely to produce results with relatively large uncertainty. Some estimate of the susceptibility of disposal mound to erosion may be made, however, from available estimates of the depth at which sand is moved by waves in these areas. Such estimates have been made from both bathymetric data and from wave observations coupled with an empirical suspension criteria for sand (Hallermeier, 1981). Along the open coast of New Jersey, extreme waves (those whose heights are only exceeded for 12 hours per year) can disturb sediments down to a depth of 7 m (23 feet). At the more protected sites, say, in the Lower Bay of New York Harbor, the disturbance should be less.

Furthermore, currents in the harbor should not be expected to cause erosion problems for deposits placed in borrow pits. In general, the configuration of the sea floor is in equilibrium with, or at least has an amicable arrangement with, the prevailing currents. For this reason, there is a legitimate concern that mounds of dredged sediment that rise above the level of the ambient sea floor may be reduced by the currents to the ambient, pre-mound levels. (Of course, this will not always happen.) On the other hand, whenever a subaqueous excavation is dredged, there is very rarely, if ever, concern that the prevalent currents will deepen the excavation. The problem is always one of shoaling in the dredged area rather than natural erosion. Dredged pits in the Lower Bay accumulate to fine-grained sediment at a rapid rate. This behavior is due to a salinity stratification that establishes itself over the pit and substantially diminishes the strength of the tidal currents within the pit.

In order to examine this behavior, salinity observations were made in two pits in New York Harbor. One of these was about 400 m (437 yards) across and the ambient sea floor is at a depth of about 9 m (30 feet). Measurements of salinity profiles over a tidal cycle show that the halocline occurred at a depth between 7 m and 9 m (23 feet and 30 feet). The stratification developed within the space of 2 m (6.6 feet) at the level of the ambient sea floor. This would suggest that the pit could be filled, at least, to within 2 m (6.6 feet) of the ambient sea floor and still retain its behavior as a sediment trap.

The second pit that was examined was about 800 m (875 yards) across in a direction approximately parallel to the tidal currents. The ambient sea floor here was at a depth of about 3.5 m (12.5 feet). Measurements of the salinity profile showed that on the floor tide, the halocline formed between 2.5 m and 5 m (16 feet and 25 feet). The stratification here appears to develop within the space of 2.5 m (8 feet) but may be up to 4 m (13 feet) below the ambient sea floor.

To generalize these observations, the aspect ratio might be used. The aspect ratio is the ratio of the pit's relief (or depth below the ambient sea floor) to the pit diameter; this particular parameter is usually used to describe the behavior of devices to trap sediment. For one pit, stratification should develop at an aspect ratio of at most 2 m/400 m or 0.005. This is an upper limit because the salinity measurements could only resolve a change within a 2-meter interval. For the other pit, the aspect ratio would be at most 4 m/800 m, or again, 0.005. An empirical rule for pits under conditions like those in New York Harbor would be that, for salinity under stratification to develop and hence for the trapping of fine-grained sediment, the aspect ratio should be at least 0.005.

Based on these considerations, the top of the fine-grained deposit in pits in New York Harbor should be approximately 23 feet (7 m) below sea level and about 6 feet (1.8 m) below the ambient sea floor. Within the uncertainties in these values, they are essentially the same as the depth limits placed on the project by the operational criteria. The final sand cap should be about 3 feet (0.9 m) thick (Bokuniewicz, Cerrato and Mitchell, 1983) so the pit floor must initially be deeper than 21 feet (19 m) in the interior. The capacity of the pit depends upon how much the actual depths exceed these limits. For a pit of the minimum radius, the capacity increases by about 200,000 cubid yards (153,000 cubic meters) for every 3 feet (0.9 m) of additional depth.

Other Considerations

Given a choice among potential sites that meet the minimum criteria, other considerations could be used to establish preferences. Briefly, these other criteria may be:

(1) More protected sites would be preferable to less protected sites.

(2) Sites with larger capacity would probably be preferable to sites with smaller capacity.

(3) Deeper sites with smaller areas would be preferable to shallower sites with larger areas in order to minimize the area of the sea floor committed to the disposal site and the volume of cap material required.

(4) Existing sites or sites that do not require modifications would be preferable to other areas in order to minimize the initial costs.

(5) Pits with steeper side slopes are preferable to those with shallower slopes.

Conclusions

Our experience with disposal operations in nearshore waters is sufficient to design criteria for borrow-pit disposal sites. Suitable pits must be more than 5 feet (1.5 m) below the ambient sea floor over an area greater than 500 yards (457 m) in diameter. Significant limitations are due to the operating requirements of the equipment rather than to the physical processes by which dredged material is placed on the sea floor. As a result, careful control of the disposal operation is essential.

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Biodata

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Dr. Bokuniewicz is an associate professor at the Marine Sciences Research Center of the State University of New York. He received his degree from Yale University in geophysics. He is primarily concerned with coastal sediment transport especially in estuaries. Over the past 15 years, Dr. Bokuniewicz has applied his research to interdisciplinary studies of problems associated with dredging and the disposal of dredged sediment.

Luncheon Address

Major General Henry Hatch

I am pleased to address the nation's premier technical group in the field of dredging. I'd say that even if Bill Murden and Chuck Hummer, two Corps "devious dredgers," didn't hold leadership positions in the organization.

Speaking of technical advanced in dredging, last week the Corps co-sponsored the first Interagency Workshop on Beneficial Uses of Dredged Material. Perhaps many of you in the audience participated. It was a nationwide forum; the Corps, other agencies, states, universities, etc. shared experiences both successful and unsuccessful in placing this material where it can do society some good. For those who weren't able to attend, or as a recap for those who did, Bo Smith of the Corps' Waterways Experiment Station is scheduled to report on the workshop later today. Stay tuned.

This morning, Dr. Dickey from the Office of the Assistant Secretary of the Army for Civil Works gave a status report on H.R. 6, the omnibus water resources authorization bill. Its status is literally changing from hour-to-hour this week as Congress finishes its business and gets ready to adjourn. It's exciting, especially since it's been 16 years since passage of the last omnibus bill to authorize new construction.

During my few minutes with you, I'll try to address the effects of H.R. 6 on the Corps Dredging Program and the dredging industry.

National Dredging Program: Scope of Work Today

The Corps maintains over 25,000 miles of waterways that serve 130 of the nation's largest cities, which reach 41 states, plus 49 major commercial ports and over 400 smaller harbors. The vast majority of inland and harbor channels require periodic dredging to provide safe, efficient conditions for maritime traffic.

The Corps now oversees dredging of about 320 million cubic yards (mcy) of sediment annually. Another 100 to 150 mcy are dredged annually by other federal and state agencies, and the private sector. Disposal is regulated by the Corps under federal legislation.

Private contractors already had a large share (about 57 percent of the contracts) of the Corps dredging work before passage of the 1978 law to reduce the Corps fleet to a minimum. The 1978 law gave new life to an industry that had been on the decline as a result of the decline in the Corps dredging program, particularly in new work. The Corps meanwhile reduced its fleet from 29 dredges to 12. In 1986 contractors moved 83 percent of the volume of material dredged; about 90 percent of the dollar value of the Corps work.

The 1978 law gave impetus for new dredging equipment. Before 1978, industry had only two hopper dredges. Since then, it has built 11 more. Meanwhile, the Corps built three state-of-the-art hopper dredges to replace older plans and provide for emergency and defense needs. Much of the new technology for both hopper and non-hopper dredges came from Europe, especially Holland. The Jones Act prohibits use of foreign-built dredges on U.S. projects, but it doesn't prohibit non-U.S. interests from selling technology or participating financially in U.S. ventures. But, nice as that new equipment is, it does no good if it sits idle in the face of a declining workload.

Dredging Needs Outlook

The volume of the Corps dredging program has been going down over the past several years. Maintenance dredging has remained fairly constant, about 260 million cubic yards a year. But new work dredging, for waterway and harbor improvement, has declined from 260 million cubic yards in 1963 to only 60 million in 1986. Practically no new major deepening projects were authorized by Congress. No bill for water resources construction has been passed in the last 16 years. This is probably due to at least two factors: (1) concern over environmental protection and (2) accommodating cost sharing. The need is especially acute in days of budget deficits.

It is likely that dredging will be significantly expanded in the near future. H.R. 6 authorizes $14 to $20 billion for water-related development. Over $2 billion is designated for port and waterways projects, mostly dredging. This includes 35 deep-draft navigation projects. One of the biggest is here in Baltimore, the 50-foot channel. An estimated cost of $370 million will require moving 66 million cubic yards of material. Another big project nearby is at Norfolk, moving 62.5 million cubic yhards at a cost of $400 million. Other big-time dredging projects included in H.R. 6: (1) Mississippi Ship channel, Baton Rouge to the Gulf: 110 million cubic yards, cost–$486 million; (2) Mobile Harbor: 102 million cubic yards, cost–$415 million; (3) Kill Van Kull, a channel between Staten Island and New Jersey: cost–$290 million. All told, 35 channel deepening projects in H.R. 6 would generate well over a billion cubic yards of new dredged material over the next decade or so.

Major breakthroughs contained in H.R. 6 are cost-sharing provisions which would require the financial participation of port authorities for a portion of new work or improvement dredging as well as maintenance dredging.

Non-federal share for new work at ports varies from 10 to 50 percent, depending on depth. We have already negotiated agreements for about a dozen ports, including the big ones I mentioned, but we must have authorities for cost sharing contained in H.R. 6 in place before we can proceed, including ad valorem fee.

On waterways, no "local sponsor" exists, but "burden sharing" takes the form of a gradual increase in waterway fuel tax, from 10 to 20 cents/gallon over the next 10 years. A conference on H.R. 6 added language for a waterway user Advisory Committee, to advise the Secretary of the Army on how the inland waterway trust fund should be spent.

The concept of "burden-sharing" was seen as radical by potential sponsors, including port authorities and waterway user groups, when first proposed. Gradually, they realized that in times of big deficits and limited federal "discretionary" funds, it represented the only way to move ahead on needed projects.

Passage of H.R. 6 with burden-sharing involved would represent the greatest cultural change in the way the Corps does business since the River and Harbor Act of 1824 got the Corps involved in navigation work in the first place. Non-federal sponsors will be involved right from the start with project planning. The system will weed out inefficient project proposals; no sponsors would be willing to sink their own money into studying them, let alone building them.

Before I leave the topic, let me emphasize that I think cost sharing is a good idea that will ultimately lead to more productive, efficient projects.

On top of new work generated by H.R. 6, there are also increased defense dredging needs. This is related to plans to expand the Navy to 600 ships. Several categories of vessels now entering the fleet may require significantly deeper access channels, more frequent maintenance dredging, or both. The Navy is embarking on its homeporting program to expand the number of types of homeports to accommodate the expanded fleet.

Dredging is also involved in waterway clean-up projects under the Superfund. In coastal areas alone, NOAA has identified over 100 site-specific clean-up requirements. Additional clean-up requirements on the waterway system seems to surface almost monthly. All such clean-up activities involving dredging will require scientific analysis of sediments, appropriate dredging technology for removal and appropriate alternatives for disposal.

Still another requirement for dredging is in removal of sediment accumulations from behind locks and dams such as effectively restore water quality, floodwater storage, hydroelectric potential, irrigation, and other project design purposes. As in other cases, we must find means to safely and economically dispose of large volumes of sediments.

The anticipated future dredging workload will require hard work and innovation on the part of everyone involved. We are looking at an estimated 50 percent increase in total dredging, a boom time for the industry. It was been frustrating to see a maritime nation such as ours agonize over development of ports and waterways. Hopefully, we can now put that behind us and get to work.

P.S. H.R. 6, the omnibus Water Resources Authorization bill, was passed on October 17, 1986.

The Success Story of Gaillard Island: A Corps Confined Disposal Facility

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Background

The Corps of Engineers has built hundreds of dredged material sites in or near U.S. waterways over many years of dredging activities. Many of these sites have provided excellent habitats for wildlife and fish, either inadvertently and not part of the project purpose, or, in recent years, through purposeful design of such sites to benefit natural resources while carrying out dredging work. For example, the Corps has built from dredged material more than 130 wetlands comprising thousands of acres, and more than 2,000 islands which are frequently used as nesting sites by waterbirds, waterfowl and other species.

Gaillard Island, built in 1980-81 by the Mobile Corps of Engineers District in lower Mobile Bay, is an excellent example of Corps efforts to incorporate beneficial uses of a dredged material confined disposal facility (CDF) while accomplishing the Corps mission of maintaining navigation in U.S. harbors and waterways. The island was built to provide a disposal site for dredged material from the deepening and widening of Theodore Ship Channel, and for Theodore Channel maintenance material. It has a projected 50- to 80-year life, depending upon level of management. Project plans have been developing since the mid-1970s, and were cumulated with island construction.

The triangular-shaped, 1,300-acre site was built at the junction of the Theodore Channel with the Mobile Ship Channel, two miles from the western shoreline of the bay (Figure 1). A secondary channel is located on the third side of the island. The island was constructed with silty sand dredged material which was hydraulically pumped using a suspended boom. Broad, gently-sloped dikes were formed using this disposal method, and the island has a large, interior containment area with approximately 600 to 700 acres of shallow water. Gaillard Island was built in an area of Mobile Bay with relatively low benthic productivity, and replaced this bay bottom habitat with a combination of island, wetland and aquatic habitats.

The three dikes are maintained and upgraded using dredged material either from maintenance dredging or borrowed from the island's interior. Construction of the island in an area with some soft foundation created a challenge to Mobile District and has been met using a variety of means. Subsidence on portions of the south dike has caused some problems, and hurricanes have overtopped portions of the island on three occasions. Dike integrity after storms has been restored using material pumped into minor breaches that occurred.

Erosion from wind fetch and ship waves has also caused some dike stabilization problems (Figure 2). Dike stabilization on the Mobile Ship Channel side (east dike) is being provided by stone armor. Stabilization on the secondary channel side (northwest dike) is provided by salt marsh, and on the Theodore Channel side (south dike) by a combination of salt marsh and stone armor.

Mobile District installed a large, ungated weir on the northern end of the east dike to allow for intertidal flow in and out of the containment area. This was done to relieve pressure on the dikes from an accumulation of rainwater and water from the dredging process.

The study of Gaillard Island has been limited by funding and manpower restraints; however, the creation and colonization of the island by vegetation and wildlife has been documented both qualitatively and quantitatively as thoroughly as possible using a low-level monitoring effort. Chronological colonization data are briefly presented in this paper, and will be the subject of a Corps technical report scheduled for publication in 1989.

Wildlife and Fish

Seabirds

Even before construction of Gaillard Island was completed, seabirds were congregating and nesting on the dikes. In 1984, 1985 and 1986, an estimated 16,000 birds nested on the island each year. This is not an unusual phenomenon for dredged material islands, and such rapid colonization of large populations has occurred on disposal sites in North Carolina, Mississippi, Louisiana, Texas, Florida, the Great Lakes, Chesapeake Bay, Columbia River and other areas (Landin, 1980, 1984 and 1986).

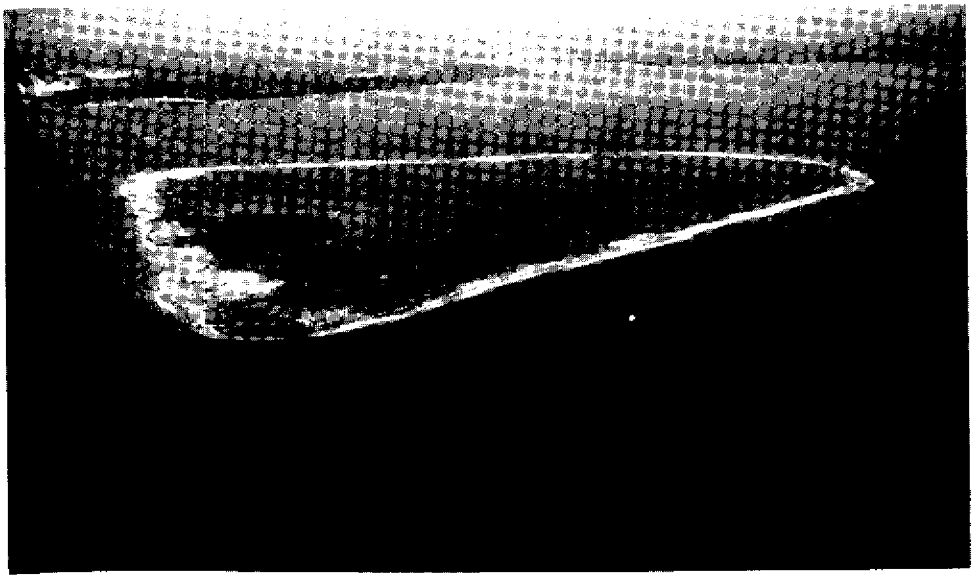

Figure 1. An aerial view of Gaillard Island CDF, Alabama, in 1983 when the island was two years old.

Figure 2. Wind fetch and ship and storm waves caused some dike erosion problems on Gaillard Island which were addressed using a combination of stone armor and wetland establishment.

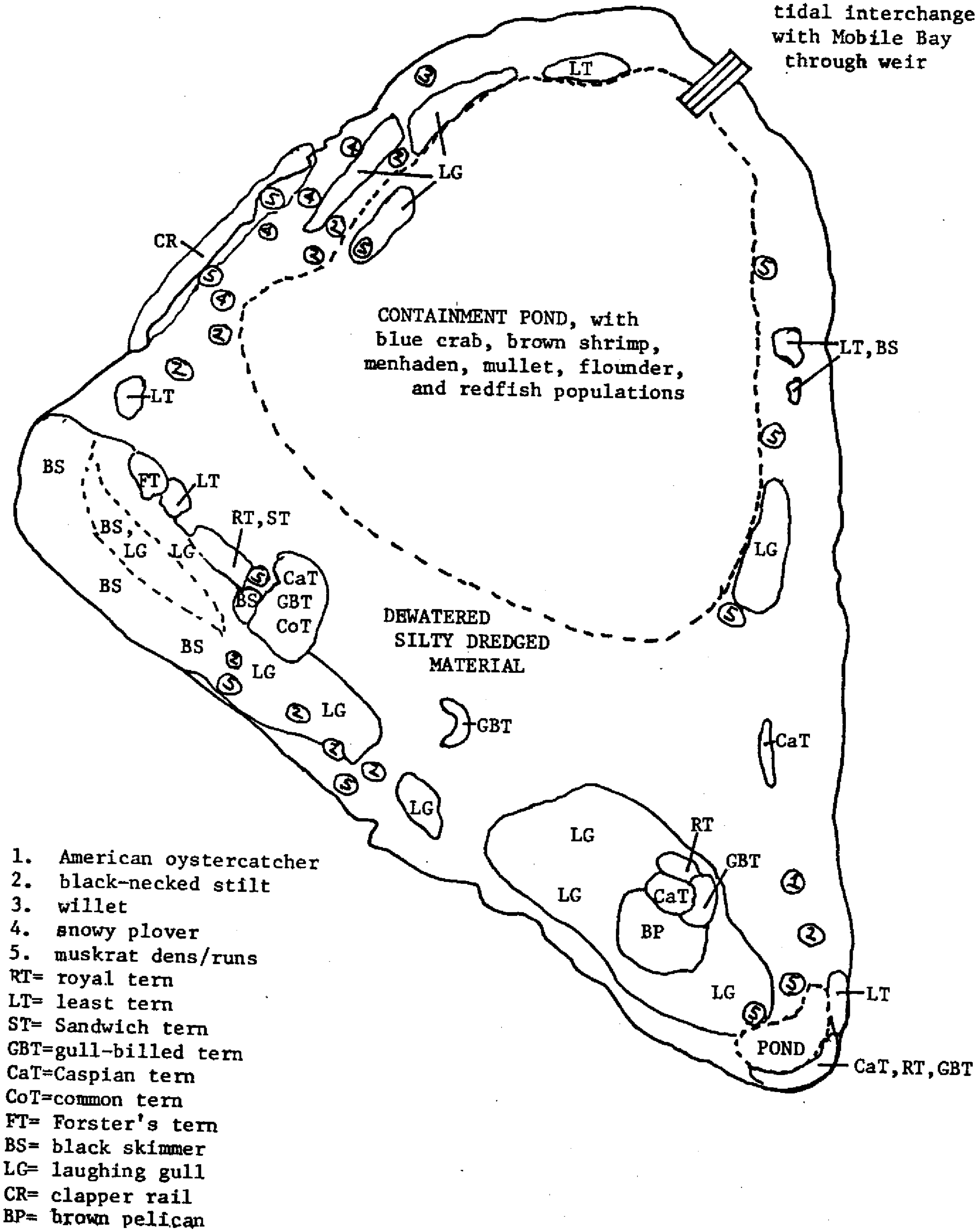

Figure 3. Location of breeding bird colonies and muskrat den/run sites on Gaillard Island in lower Mobile Bay, Alabama, in June 1986.

An estimated 4,000 laughing gulls, black skimmers and terns were nesting in 1981. An estimated 7,000 birds of the same species nested in 1982, and have nested in increasing numbers each year. Table 1 lists nesting species on Gaillard Island, the year in which nesting first occurred, and nesting estimates. A schematic of the three dikes on the island, showing colony locations for 1986, is shown in Figure 3. Nest counts were made each year using one of two methods. In colonies with low numbers of nests or where the data on the species was considered critical (endangered or rare), ever nest was counted. In colonies with very large numbers, a 30-foot-wide belt transect was walked through the colony, in which each nest was counted. An estimate of number of nests was then determined by measuring the size of the colony area and extrapolation. Numbers of eggs and chicks in each counted nest were noted, and averages for eggs/chicks per nest were determined. No attempt to record data for dates of egg-laying and incubation, for chick survival or for fledgling rates was made. An intensive monitoring effort would be necessary for this information to be gained. Not only would it have resulted in undue disturbance of nesting colonies at a critical time, but neither time nor funds were available for more frequent sampling.

Black skimmers have increased in 1986 to an estimated 3,500 birds nesting, the largest black skimmer colony on the northern Gulf coast (Figure 4). Over 12,000 laughing gulls nested on Gaillard Island in 1985. The number of gull nests dropped slightly in 1986; however, an increase in both number of other seabird species and individuals within other species was observed. Since gulls are predators on tern eggs and chicks, the decrease in gull nesting was considered a benefit to tern species.

Seven species of terns (least, Caspian, Royal, common, Forster's, gull-billed, and Sandwich) were nesting in 1986. The 194 nests of least terns were a great increase over the previous years, and least tern colonies on the 1,300-acre island increased from one in 1985 and prior years to five in 1986.

Abundant tern, skimmer and gull habitats are available on Gaillard Island. Caspian, royal, Sandwich, gull-billed, and least terns nest on bare or nearly bare areas on the island (Figure 5), while common, gull-billed and royal terns and black skimmers nest in sparse herbaceous cover. Forster's terns and laughing gulls nest in the dense herbaceous cover on the island, especially along the south dike and a portion of the northwest dike.

Some gull-billed, royal and Caspian terns nested on the fine-textured silt dredged material inside the dewatered portion of the containment area where desiccation cracks were less distinct. Chicks clambered in an out of these shallow cracks as they moved about the colony with no apparent injury. Gull-billed terns collected small oyster shell fragments for their nests, and laid their eggs on these small mounds. Some black skimmers also nested on the inside of the containment area on well-drained dewatered silty sand. However, the largest skimmer concentrations were on the outer south dike slopes.

Pelicans

Within a year of island construction, both brown and while pelicans were using the containment area for loafing and feeding on a year-round basis. Non-breeding white pelicans have remained on Gaillard Island but have not made an attempt to nest. Both species feed inside the containment pond, especially at the weir where fish move in and out of the pipes.

Brown pelicans built four nest on the east dike in June 1983. One nest was successful, and two chicks fledged. This range expansion brought nesting brown pelicans back into Alabama for the first time in this century. In 1984, eight nests were successful, and in 1985, 133 nests fledged over 250 chicks. This remarkable increase in colony size was further enhanced by over 200 nests in 1986 in which over 500 chicks fledged (Figure 6). There was an average of 2.3 chicks per nest. In June 1985, over 300 subadult and adult brown pelicans were observed on the island, with approximately 500 chicks and eggs in nests.

When brown pelicans began nesting on Gaillard Island, they were still listed as endangered. The U.S. Fish and Wildlife Service has downgraded their species' status on the Atlantic and Gulf coasts. In 1985, based on the one colony on Gaillard Island, the state of Alabama also removed the brown pelican from its endangered species list. This is the only brown pelican colony between south Florida and south Louisiana, and these delistings may be premature.

Other Bird Species

By 1982, herons and egrets from the Alabama mainland had discovered the feeding areas inside Gaillard Island (Figure 7). These consisted on four habitats: (a) the ponded brackish swales created from subsidence and sand accretion; (b) the shallow water of the estimated 600- to 700-acre containment pond; (c) the borrow pits created from dike upgrading; and (d) the planted intertidal marshes. Species observed using island habitats include great blue heron, little blue heron, tri-colored heron, common egret, yellow-crowned night heron, and snowy egret. No nesting by herons and egrets has occurred, but nesting substrates for these

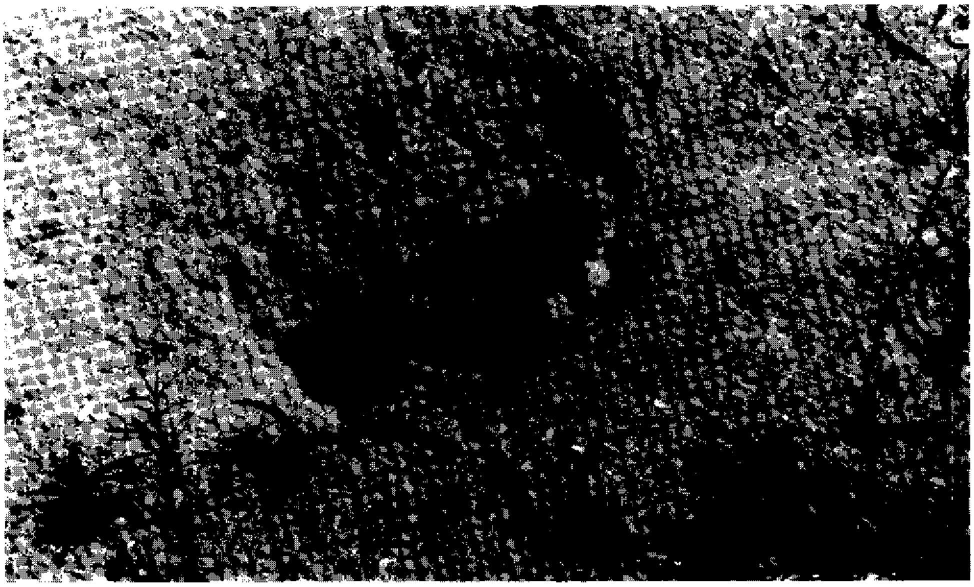

Figure 4. A nest of black skimmer chicks on Gaillard Island. Skimmers have nested there each year since 1981, and in 1986 an estimated 3,500 skimmers nested on the south and east dikes.

Figure 5. Royal terns have nested on Gaillard Island every year since 1981, and prefer bare ground habitat.

Figure 6. *The brown pelican colony on Gaillard Island in 1986, which contained over 200 nests.*

Figure 7. *Egrets and herons fly from the mainland to feed in swales, shallows and borrow pits on Gaillard Island.*

species are still developing on the island. It is entirely possible that nesting will occur when vegetation conditions are suitable (large shrubs and trees).

Other waterbirds are frequently observed on the island, and include nesting black-necked stilts (11 pairs in 1986), and clapper rails, which have increased in numbers each year of the island's existence. Black-necked stilts primarily nest on the edges of vegetated swales and borrow pits, while clapper rails nest only in the planted salt marsh.

Shorebirds have used Gaillard Island habitats during migration and overwintering since the island was under construction. During spring and fall migrations, thousands of these birds can be observed feeding on mud flats inside the containment area and along the shoreline. In addition to this very heavy use, willets, American oystercatchers and snow plovers nest on the island.

Waterfowl also use the containment area for feeding and resting, with considerable overwintering use by lesser scaup, ruddy ducks and other divers, mallards and black ducks. Mottled ducks nest on dredged material islands and in natural marshes along the Gulf coast, but have not yet been observed nesting on Gaillard Island.

Only a few songbirds have been observed on the island. Nesting species include marsh wrens and seaside sparrows in the salt marsh, and common grackles, boat-tailed grackles and red-winged blackbirds in small trees and vines on the higher portions of the dikes. Barn and other swallows in small numbers have been observed feeding over the containment pond during migration.

Muskrats

In 1985, muskrats colonized Gaillard Island. Although their original location is unknown, they could have floated to the island on driftwood from the rivers feeding the bay or possibly could have swum the two miles from shore. Enough muskrats were on Gaillard Island by mid-1986 to populate vegetated areas on all three dikes. They have built runs and dens on the dikes and around the swales and borrow pits. One muskrat mound was found in 1986 in a south dike swale, while the rest of the growing population lives in underground dens. Since muskrats feed almost exclusively on vegetation, especially saltmarsh bulrush (*Scirpus robustus*) and American three-square (*Scirpus americanus*), they are not considered a threat to the nesting seabirds.

Aquatic Biota

The low level of monitoring at Gaillard Island did not include quantitative data collection on aquatic biota. Observations of presence and abundance are purely qualitative, and based on such factors as feeding observations of pelicans and other seabirds and the increase in nesting and successful fledging. They are also based on reports, observations, and interviews with commercial and sports fishermen, crabbers and shrimpers. One commercial crabber reported daily catches of 120 to 200 pounds of blue crabs inside the containment pond in 1985, and he has been crabbing inside the dikes for at least three years. Catches of mullet, menhaden, redfish, flounder and brown shrimp have also been observed or reported. Amateur crabbers frequent the shallow waters of the containment pond.

Vegetation

The first vegetation to appear on Gaillard Island occurred within months after the island was built, with the occurrence of a few weedy species such as dog fennel (*Eupatorium capillifolium*), the nesting substrate used by the brown pelicans two years later. Natural colonization steadily increased since 1981, but has not occurred as rapidly as a disturbed soil or disposal site located closer to or on the mainland. Soil salinity may have slowed colonization in early months. However, high precipitation in the Mobile area coupled with moderately well-drained silty sand dredged material allowed freshwater and brackish plant species to colonize and grow within a few months of island creation.

Large portions of the three dikes, especially the south and northwest dikes, are nearly completely covered with dense herbaceous vegetation. Plant species colonizing the island benefited from planted areas which provided protection and substrate for their seeds and other propagules.

Planted Wetland Areas

Beginning in 1981, Mobile District and Waterways Experiment Station carried out a series of dike stabilization experiments in moderate wave energies at Gaillard Island, in which smooth cordgrass (*Spartina alterniflora*) was planted in the intertidal zone on the entire northwest dike and potions of the south dike. These plantings were coupled with low-cost erosion control structures and features to provide temporary protection to the planted marsh. In 1981-1983, fixed and floating tire breakwaters were used as erosion control structures. These were tested in wave energy models at Waterways Experiment Station, and the best breakwater configurations were used in field experiments. Breakwaters were anchored in front of the planted marsh to break wave actions, and cost approximately one-fourth the cost of conventional stone armor placement (Allen et al., 1983).

In 1983-1986, experimental plots were planted, coupled with a variety of even less costly techniques (one-tenth to one-fourth less than stone armor). Smooth cordgrass sprigs were planted in burlap plant rolls, in various thicknesses of erosion control mat (Paratex), in grid mattress, and in anchored tires belted together across the intertidal area (Allen et al., 1984; Webb et al., 1984). The burlap plant rolls and 3-inch thicknesses of erosion control mat provided the most stability for the new transplants of smooth cordgrass while they were establishing (Allen et al., 1986). These later tested techniques worked as effectively as the more expensive floating tire breakwaters. Control areas were also planted each year so that a valid statistical comparison could be made. Details of these experiments are presented in Allen et al. (1983, 1984 and 1986), where specific information on each technique is available.

In spite of the washout of some plant propagules from storm and wave action, by 1986 the northwest dike intertidal area had been effectively stabilized from both replanting of washout areas and from spread of surviving sprigs throughout the planted area (Figures 8 and 9). On the south dike a combination of both washout and subsidence destroyed the first plantings (in 1983). Subsequent test plots have been somewhat successful. However, wave action and wind fetch are greater on the south dike than on the northwest dike, and erosion problems on most of the south dike cannot be solved readily using existing biostabilization technology. At the present time, the combination of planting and stone armor seems to be stabilizing that dike.

An interesting feature of the planted salt marsh is that it traps large quantities of sand from the bay. After winters in which smooth cordgrass has died back due to cold weather, and sand has simultaneously accumulated, portions of the salt marsh appear to be smothered. However, each year the marsh has grown through the sand berm which formed and has grown farther out into the bay. This has expanded the width of the marsh and increased the stability of the northwest dike.

In conjunction with this sand accumulation, swales have formed behind the berms. These swales have colonized with brackish marsh plants, primarily American three-square, saltmarsh bulrush and southern cattail (*Typha domingensis*). Propagule sources for these species were marshes on the mainland over two miles away. On the south dike where subsidence occurred, resulting brackish ponds also colonized with these same species.

Table 2 shows plant species occurring on Gaillard Island, and notes whether the species was planted or occurred naturally, the year it first appeared on the island, and the habitat in which it grows. Landin (1986) and EM 1110-2-5026, Beneficial Uses of Dredged Material, are other sources of information on colonization of dredged material islands.

Planted Upland Areas

In 1982, Mobile District had the island dikes aerially seeded with a mixture of grasses, including common Bermuda grass (*Cynodon dactylon*), barnyard grass (*Echinochloa crusgalli*), and common crabgrass (*Digitaria sanguinalis*). These plants initially grew on dike slopes, especially in piles of driftwood and wrack formed from wave action and storm tides. By 1984, dike crests on undisturbed portions of the dike had complete plant covers largely dominated by common Bermuda grass. These grass stands were heavily mixed with colonizing species of trailing wildbeam (*Strophostyles helvola*), yankee weed (*Eupatorium compositifolium*) dog fennel and sea purslane (*Sesuvium portulacastrum*). On dike areas where upgrading and stabilization work was necessary, vegetation was covered over, and these areas have revegetated over a period of 1 to 2 years, with the dike crests taking much longer than the dike slopes to recover.

In 1983, Mobile District contracted to have trees planted on the island, and a variety of native and exotic species were transplanted on the dikes. Species native to the Gulf coast are surviving and growing quite well, with some pine trees already 10 feet tall. Exotic species and those not acclimated to a Gulf coast climate have died. Trees on the east dike and in areas where upgrading of the dikes was necessary have been covered over. After dike stabilization is complete, portions of the upland areas may be replanted with native coastal species to provide woody vegetation on the island.

Summary

Gaillard Island, now five years old, replaced 1,300 aquatic acres of Mobile Bay with a combination of island, wetland and aquatic habitat. The island CDF has increased significantly each year in its value for wildlife and fish, while providing a long-term containment site for dredged material from Theodore Channel. It has provided a testing site for important wetland development studies using biostabilization techniques, and has provided

Figure 8. Sprigs of smooth cordgrass two months after planting on the northwest dike of Gaillard Island.

Figure 9. The same smooth cordgrass marsh on the northwest dike of Gaillard Island three years after planting.

highly important nesting habitat contributions for seabirds on the northern Gulf coast. Gaillard Island has proven to be such an excellent site for natural resources through the use of dredged material that the Chief, U.S. Army Corps of Engineers, selected the island to receive the Environmental Design Award for 1985. As this CDF develops, its potential for greater natural resource beneficial use will increase. Management of the island's interior will allow continued aquatic habitat for a number of years, while management and protection of the upland and wetland areas will allow increased use of the site by nesting seabirds and feeding waterbirds, waterfowl and shorebirds. This should be concurrent with engineering management techniques which will add to the active life of the island for disposal of dredged material.

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Biodata

Mary C. Landin

Dr. Mary C. Landin has conducted research on dredging problems with regard to beneficial use applications and natural resource impacts for the past 13 years. She has written numerous technical reports and papers on dredging, including the new Engineer Manual on Beneficial Uses of Dredged Material. She says the part of her work she enjoys most is the day-to-day consultations and interactions on dredged material disposal opportunities and problems with Corps districts and divisions, other agencies and the public. She is responsible for much of the state-of-the-science information on habitat development using dredged material, especially in wetlands and on islands.

Table1. Nesting Species on Gaillard Island Confined Disposal Facility

SPECIES	NUMBER OF NESTS 1981	1982	1983	1984	1985	1986
American oystercatcher (*Haematopus palliatus*)	—	—	—	—	—	1
Black-necked stilt (*Himantopus mexicanus*)	—	1	2	4	7	11
Black skimmer (*Rynchops niger*)	500*	800*	1200*	1575*	1500*	1750*
Boat-tailed grackle (*Quiscalus major*)	—	—	—	1	1	1
Brown pelican (*Pelecanus occidentalis*)	—	—	1	8	133	224
Caspian tern (*Sterna caspia*)	—	—	50*	50*	75*	63
Clapper rail (*Rallus longirostris*)	—	—	1	1	2	2
Common grackle (*Quiscalus quiscula*)	—	—	—	—	1	4
Common tern (*Sterna hirundo*)	—	—	—	—	—	7
Forster's tern (*Sterna forsterii*)	—	—	6	12	13	9
Gull-billed tern (*Sterna nilotica*)	—	—	—	20	35	42
Laughing gull (*Larus atricilla*)	1500*	3000*	4500*	6000*	6250*	5500*
Least tern (*Sterna antillarum*)	22	14	19	27	40	194
Marsh wren (*Cistothorus palustris*)	—	—	1	2	2	3
Red-winged blackbird (*Agelauis phoeniceus*)	—	—	—	1	3	9
Royal tern (*Sterna maxima*)	23	35*	40*	50*	63	74
Sandwich tern (*Sterna sandvicensis*)	—	1	1	3	1	2
Seaside sparrow (*Ammospiza maritimus*)	—	—	1	1	2	2
Snowy plover (*Charadrius alexandrinus*)	—	—	—	—	—	4
Willet (*Catophrophorus semipalmatus*)	—	1	1	1	1	1
Nest Totals	2045	3852	5823	7756	8129	7912

*Nest numbers were estimated in larger colonies.

Table 2. Plant Species Occurring on Gaillard Island, Mobile Bay, Alabama

SPECIES	Year First Occurred	Means of Occurrence	Remarks
Alligator weed (*Alternanthera philoxeroides*)	1982	colonized	uncommon
American sycamore (*Platanus occidentalis*)	1982	colonized	uncommon, stressed
American three-square (*Scirpus americanus*)	1982	colonized	scattered stands
Bahia grass (*Paspalum notatum*)	1982	seeded	common, abundant at some locations
Baldcypress (*Taxodium distichum*)	1982	planted	uncommon, stressed
Barnyard grass (*Echinochloa crusgalli*)	1982	seeded	common, abundant inside dikes in low-lying areas
Beach morning glory (*Ipomoea stolonigera*)	1983	colonized	uncommon
Beach panic grass (*Panicum amarulum*)	1982	colonized	common
Big cordgrass (*Spartina cynosuroides*)	1983	colonized	scattered stands
Big smartweed (*Polygonum pennsylvaincum*)	1983	colonized	uncommon
Bitter mint (*Hyptis alata*)	1984	colonized	uncommon
Bitter panic grass (*Panicum amarum*)	1982	seeded	scattered stands
Black needlerush (*Juncus roemerianus*)	1985	colonized	uncommon in low-lying areas
Black willow (*Salix nigra*)	1982	colonized	isolated small trees
Broom sedge (*Andropogon virginicus*)	1983	colonized	common
Browntop millet (*Echinochloa walterii*)	1984	colonized	uncommon
Cabbage palm (*Sabal palmetto*)	1983	planted	stressed; dead
Chufa (*Cyperus esculentus*)	1984	colonized	scattered plants
Chinese tallow (*Sapium sebitiferum*)	1983	planted	stressed
Cocklebur (*Xanthium strumarum*)	1984	colonized	scattered plants
Colorado river hemp (*Sesbania portulacastrum*)	1985	colonized	uncommon
Common Bermuda grass (*Cynodon dactylon*)	1982	seeded	abundant on all undisturbed dikes
Common crabgrass (*Digitarias sanguinalis*)	1982	seeded	common
Common purslane (*Portulaca oleracea*)	1983	colonized	uncommon
Common ragweed (*Ambrosia artemisiifolia*)	1982	colonized	common on all dikes
Common reed (Phra*gmites australis*)	1982	planted	small to large stands on all dikes
Dallis grass (P*aspalum dilatatum*)	1983	colonized	uncommon
Dandelion (*Taxarcum officinale*)	1984	colonized	uncommon
Day flower (*Commelina sp.*)	1985	colonized	uncommon
Dog fennel (*Eupatorium capillifolium*)	1981	colonized	common, abundant in some nesting areas
Eastern baccharis (Bacc*haris halimifolia*)	1983	colonized	uncommon
Eastern red cedar (*Juniperus virginiana*)	1983	planted	stressed, uncommon
Eurasian water-milfoil (*Myriophyllum spicatum*)	1984	colonized	uncommon
Fall panic grass (*Panicum dichotomiflorum*)	1982	seeded	common, abundant in some areas on dikes
Giant reed (*Arundo donax*)	1983	planted	uncommon
Globe nutsedge (*Cyperus globusus*)	1982	colonized	common on all dikes
Goosefoot (*Chenopodium ambrosioides*)	1984	colonized	common on NW dike
Green ash (*Fraxinus pennsylvanicum*)	1983	colonized	stressed, dead
Ground nut (Apio*s* sp.)	1984	colonized	uncommon
Horse nettle (*Solanum carolinense*)	1983	planted	stressed, dead
Japanese pittisporum (*Pittisporum tobira*)	1983	planted	stressed, dead
Jewelweed (*Impatiens pallida*)	1984	colonized	uncommon in wet areas

continued

SPECIES	Year First Occurred	Means of Occurrence	Remarks
Johnson grass (*Sorghum halepense*)	1985	colonized	uncommon
Knotroot bristlegrass (*Setaria geniculata*)	1983	colonized	common
Leafy three-square (*Scirpus maritimus*)	1985	colonized	uncommon in wet areas
Live oak (*Quercus virginiana*)	1983	planted	transplants surviving
Longleaf pine (*Pinus palustris*)	1984	planted	transplants surviving
Marsh fleabane (*Pluchea* sp.)	1983	colonized	uncommon
Mimosa (*Albizzia julibrissin*)	1983	planted	stressed, dead
Nutsedges (*Cyperus* spp.)	1982	colonized	common
Nuttall's oak (*Quercus nuttallii*)	1983	planted	stressed
Parrot feather (*Myriophyllum* sp.)	1985	colonized	uncommon in wet areas
Peppergrass (*Lepidium virginicum*)	1985	colonized	uncommon on dikes
Pokeweed (*Phytolacca americanus*)	1984	colonized	uncommon
Red rattlebox (*Sesbanbia punicea*)	1983	colonized	uncommon
Rose mallow (*Hibiscus* sp.)	1986	colonized	five plants
Saltgrass (*Distichlis spicata*)	1981	colonized	common on all dikes
Saltmarsh aster (*Aster tenuiifolius*)	1982	colonized	common on all dikes
Saltmarsh bulrush (*Scirpus robustus*)	1981	colonized	abundant in wetlands
Saltmarsh sand spurry (*Spergularia marina*)	1982	colonized	uncommon
Saltmarsh morning glory (*Ipomoea sagittata*)	1982	colonized	uncommon
Saltmeadow cordgrass (*Spartina patens*)	1982	colonized	common in wetlands
Sand bur (*Cenchrus incertus*)	1985	colonized	uncommon
Sandgrass (*Triplasis purpurea*)	1982	colonized	common in some dike areas
Saw grass (*Cladium jamaicensis*)	1985	colonized	uncommon in wetlands
Sea oxeye (*Borrichia frutescens*)	1984	colonized	uncommon
Sea purslane (*Sesuvium portulacastrum*)	1981	colonized	common
Seaside goldenrod (*Solidago sempervirens*)	1982	colonized	common
Seaside heliotrope (*Heliotropium currassivicum*)	1985	colonized	uncommon on dikes
Sedges (*Carex* spp.)	1982	colonized	uncommon on dikes
Slash pine (*Pinus elliottii*)	1983	planted	transplants surviving and growing
Slender arrowhead (*Sagittaria graminea*)	1985	colonized	uncommon in wetlands
Smartweeds (*Polygonum* spp.)	1981	colonized	common on all dikes
Smell melon (*Curcubita vulgaris*)	1982	colonized	uncommon, east dike
Smooth cordgrass (*Spartina alterniflora*)	1981	planted	abundant in wetlands
Softstem bulrush (*Scirpus validus*)	1983	colonized	scattered stands
Southern cattail (*Typha domingensis*)	1982	colonized	common in wetlands
Southern magnolia (*Magnolia grandiflora*)	1983	planted	stressed, dead
Sow thistle (*Sonchus oleraceus*)	1984	colonized	uncommon on dikes
Sprangle top (*Leptochloa fascilularis*)	1983	colonized	common on dikes
Sweet gum (*Liquidambar styraciflua*)	1983	planted	stressed but growing
Trailing wildbean (*Stophostyles helvola*)	1982	colonized	common, abundant on south dike
Vasey grass (*Paspalum urvillei*)	1985	colonized	uncommon
Water hemp (*Amaranthus cannabinus*)	1984	colonized	common on dikes
Water hyacinth (*Eichhornia crassipes*)	1983	colonized	uncommon on beaches
Watermelon (*Citrullus vulgaris*)	1982	colonized	uncommon on dikes
Water smartweed (*Polygonum punctatum*)	1984	colonized	uncommon in wetlands
Water purslane (*Ludwigia palustris*)	1983	colonized	uncommon in wetlands
Water willow (*Justicia americana*)	1984	colonized	uncommon
Wax myrtle (*Myrica cerifera*)	1982	colonized	also transplanted, growing well

continued

SPECIES	Year First Occurred	Means of Occurrence	Remarks
Widgeongrass (*Ruppia maritima*)	1984	colonized	uncommon in containment pond
Wild carrot (*Daucus carota*)	1985	colonized	uncommon on dikes
Wild lettuce (*Lactuca canadensis*)	1984	colonized	uncommon on dikes
Yankee weed (*Eupatorium compositifolium*)	1982	colonized	common, abundant in some areas
Yellow nutsedge (*Cyperus rotundus*)	1983	colonized	common on dikes

Proposal to Reduce Dredge Litigation

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Abstract

A high percentage of Army Corps of Engineers dredging contracts result in contractor claims and/or litigation. Many of the claims have to do with the geotechnical aspects of the projects, and, since it is never practical to achieve a 100-percent soil sample, the elimination of all claims is impracticable. However, claims should normally be settled without recourse to litigation or arbitration.

The failure to settle a claim is generally the result of the failure to agree on:

a) the soil conditions encountered vs. those anticipated from the soil test data;

b) what constitutes a differing site condition;

c) the effect of the conditions encountered on dredge capability.

As a result of years of dredge development and analysis, plus experience as an expert witness in dredge litigation, the author sets forth a proposal calculated to reduce litigation substantially.

Proposal to Reduce Dredge Litigation

Within the last few years, I have served as a dredge expert witness in cases of arbitration or litigation against a municipality, an engineering firm, a general contractor and my government, specifically the Army Corps of Engineers. Assisting litigation against your government creates some inner turmoil, in that the witness is assisting in the prosecution of a case against himself and his fellow taxpayers. Ideally, a witness would prefer to side with his government, but a higher principle must prevail, and that is simply what is right and equitable. An expert witness should be prepared to serve objectively on either side of a case, but where his client's case lacks merit, he should recommend settlement and, under some circumstances, consider withdrawal from the case.

The Problem

All cases in which I have been involved have had two things in common: (1) an appallingly high expense, and (2) a feeling the case should have been settled out of court at great savings. When one considers the extent of involvement of top management and key personnel on both sides, the cost in terms of loss of other business is obvious. Further, there are often two or three lawyers on each side, and a similar number of expert witnesses. When the case is at its peak, these eight to twelve people can cost $8,000 to $12,000 per day plus expenses. Since it is not unusual for litigation to run three to five years, the cost of litigation can reach several hundred thousand dollars or even higher for complicated, extended trials.

What is the frequency of claims, arbitrations or litigation on dredge projects? The answer is: dismayingly frequent. I do not have the records on the recently completed Tombigbee Waterway project, but the industry is well aware that many claims were made, all of which were not settled by friendly discussion. Those of us aware of the conscientious efforts by the Corps to prepare the project, and of the experienced and competent dredging contractors involved in claims, find it difficult *not* to recognize the existence of a serious industry problem. If we can solve or even alleviate this problem, we can increase contractors' profits, reduce Corps' costs, and thus increase the number of projects fundable from the limited federal budget. A proposal calculated to accomplish a significant improvement in the economics of the dredging industry is the purpose of the report.

Causes of the Problem

Dredging projects necessitate the removal of sub-surface soil, which can't be seen and about which little is known. Even a thorough, conscientious effort by experienced drillers touches upon only a small fraction of one percent of the soil to be removed. With the vagaries of natural deposition of soils, earthquakes, volcanos, and the meandering of rivers across their flood plains over the centuries, there is small wonder that surprises are frequently in store for the contractor and the owner. So, the sub-surface soil, never completely predictable by even the best drilling program, is the first and major cause of claims.

Soil sampling problems have long been recognized and the Corps and other owners have attempted various methods of minimizing the problem. Prior to World War II, it was not uncommon to request bids on a project with the responsibility for determining soil conditions resting with the bidder. This resulted in high costs and limited bids; so the modern practice was changed to provide borings on a reasonable spacing with a description of each soil encountered as well as the blow count (a measure of hardness of the soil). This information serves to provide the bidder with a basis for his bid. However, no one, including the Corps, suggests that this tiny sample is always representative of the sub-surface soil, and, therefore, the Corps customarily includes a "Changed Condition" clause in its contracts. The purpose of this clause is to allow each bidder to submit his most competitive bid based upon the boring data provided. Then, if the boring data is not truly representative, e.g., excess gravel is encountered, the bidder can request a reasonable contract adjustment on the basis of "Changed Conditions."

This principle is conceptually sound, and is subscribed to by both Corps and dredge company personnel. However, in practice, it frequently founders because of disagreements between the Corps and bidder on the cost effect of the changed conditions, and even whether a changed condition was encountered. This form of disagreement represents the second cause of claims.

A third cause of claims is a basic disagreement between the Corps and the contractor as to the capability of the dredge, with particular emphasis on the project conditions encountered. Both the Corps and the contractor create estimates for the project, with each feeling his estimate is sound. When the actual time and cost exceed the Corps' estimate, there may be criticism of the contractor's equipment, operation and maintenance in order to contest a claim. When the actual time and cost exceed the contractor's estimate, he is not surprisingly convinced the cause is changed conditions. While it is not necessary to reconcile the two estimates, the differences in the estimates may be a manifestation of a different evaluation of the effects of various conditions on dredge productivity, and this can carry throughout the negotiations to litigation. This difference of opinion may indicate a need for a better understanding of the fundamentals of hydraulic dredging on the part of either the plaintiff or defendant or both.

Need to Resolve

Public Law 95-269, passed in the 1970s, directed the Corps to reduce its dredging fleet to a mere handful of dredges, restricted largely to emergency and demonstration purposes. Since dredging is an essential industry, vital to the U.S. commercial and defensive interests, this legislation is of landmark importance. It places the dredging industry in a cardinal role, requiring a healthy and viable industry at all times. To have a portion of the industry constantly caught up in conflicts with its major customer, the Army Corps of Engineers, is not conducive to the nation's best interests.

Obviously, any reduction in dredging claims, arbitration and litigation will be helpful. Based upon my experience, I feel a reasonable target is a 50-percent reduction in claims, and a 75-percent reduction in arbitration and litigation. If this sounds high or overly optimistic, consider the following proposals. Then, let us agree, at least, on the desirability of such a reduction as well as the need to institute a program to achieve it.

Proposals

Median Grain Size – Hydraulic dredging involves the transport of solids by entrainment in a high-velocity stream of water. Fundamental to the requirements of such a system is the nature of the solids to be transported, primarily the median grain size, or d_{50}, as shown on the grain size distribution chart for each material to be dredged. Therefore, for each rheologically distinct type of material, i.e., how it acts or flows in a water slurry, the d_{50} should be given, or better yet, the grain distribution chart provided.

On the Tombigbee project, the Corps did not make it a practice to provide median grain size or distribution charts, but rather described the material is such terms as fine to medium sand, with a "trace," or "little," or "some" gravel. The Corps uses the rather broad terms of the Unified Soils Classification System as their basis, a system that is not particularly appropriate for dredging. For example, the U.S.C.S. describes sand sizes as varying from .074 mm to 4.76 mm, or a range of 64:1. The largest d_{50} of sand is slightly less than 1/5 inch, a size most dredgemen would call pea gravel, not sand. When one realizes that 50 percent of the particles could be larger than 4.76 mm and the material still defined as sand, one begins to realize the problems with a system using word descriptions of dredged materials based upon the U.S.C.S.

Note that it is not necessary to change the U.S.C.S., which is an entrenched and utilitarian system. The special need of the dredging industry is to determine the d_{50} and/or grain distribution chart that is determining in hydraulic transport. With such information, it becomes unimportant whether a material is called fine, medium or coarse sand, for the dredgeman can then refer to his velocity vs. d_{50} chart to ascertain the velocity required for efficient transport.

Most dredge estimates recognize only a few rheologically distinct soils, such as clay, fine sand, medium sand, coarse sand and gravel and perhaps some combinations. The grain distribution charts on most projects would probably represent only ten to twenty ranges of d_{50} which would allow the dredge estimator to group them as he sees fit. For the drillers to provide samples for laboratory determination of grain distribution for each distinct type of soil would not be a major effort. Present procedures call for a split spoon sample of each type of material with detailed word descriptions, so the grain distribution chart represents a small increased effort. Clay and silt determination would not change.

Sampler Equipment – Most borings are taken using the ASTM's Standard Penetration Test, D1586-67. This S.P.T. uses a split spoon sampler with an outside diameter of 2 inches and an inside diameter of 1-3/8 inch. It is driven into the soil by a 140-pound weight falling 30 inches. The number of blows per foot is designated as the "N" value of the soil and affords some measure of the difficulty of excavation or cutting of the soil. For every distinct soil stratum the sampler is raised, and the split tube separated to disclose a relatively undisturbed soil sample for analysis.

Since many claims by dredgemen involve gravel, the shortcomings of the S.P.T. can be readily detected. Obviously, no particle larger than 1-3/8 inch will be detected in the sample while the U.S.C.S. describes gravel as extending in size up to 3 inches and cobbles up to 40 inches.

Split tube samplers are provided in standard sizes ranging from the S.P.T. 2-inch OD, 1-3/8-inch ID, to 4-1/2-inch OD, 4-inch ID. This latter size can pick up a particle almost three times as large as the S.P.T. 2-inch unit and should be used. The increase in cost of driving the larger sampler would appear miniscule compared to the credibility of the data and the money to be saved by reduced claims. On Tombigbee, there were 156 borings for a 17-mile stretch of dredging, or about one every 600 feet. One 4-1/2-inch boring based upon 300-foot centers has the potential of providing vastly improved data. The more accurate the data, the more accurate the dredging estimate and, quite probably, the fewer claims to be submitted.

Dredging Estimates – Dredge technology has progressed significantly in the last quarter century, and the capacity of a dredge can be projected with a high degree of accuracy if all the job conditions are known. The Corps presently uses a system highly dependent upon the experience and capability of the individual using the guidelines. Regulation No. 1110-2-1300 Government Estimate...For Dredging is subject to error since the d_{50} of the basic material in the production rate table is not defined; the digging depth is assumed constant; the horsepower is assigned arbitrarily on the basis of dredge size; neither coarse sand nor gravel is identified as a dredged material, nor is a multiplier assigned to modify the production rate table as is done for other material; cutter hp is not assigned or capability defined. Doubtless, in years past the system served the Corps well through its experienced dredge people. Since P.L. 95-269 was implemented and many experienced people were lost through retirement, the Corps is perhaps ready for a new, simpler and more accurate system usable by the less experienced estimator. A chart showing production for each size dredge and each job variable such as d_{50}, digging depth, line length, etc. could be provided with modest effort. Factors such as bank height, blow count, downtime, trash, etc. could be described and their effect on production rates assigned in coefficient form. This improved estimating technique could result in better evaluation of bids and the potential elimination of some claims by better prediction of performance.

Quantifying Job Variables – Both the Corps and the bidder have had to identify the job variables in order to complete their job estimate. The specs and plans have had to be examined to determine average values for line length from the dredging site to the nearest disposal site; the digging depth; the material d_{50}; the bank height; the blow count; trash stoppages; and operating time per day. If the Corps determines that an inexperienced bidder is low, but grossly out of line with his quantification of the variables, his bid could be rejected in favor of a more knowledgeable bidder. If, on the other hand, the low bidder insists that his bid should stand, he would in effect be denying his right at a later date to make a claim based upon conditions which had been called to his attention in advance. The courts have established that a successful claim requires: (1) that actual project conditions must differ materially from those *reasonably* estimated by the bidder from the project data supplied; (2) the bidder must have *reasonably* relied upon the project data; and (3) the different conditions must have been *reasonably* unforeseeable by the bidder. Identifying and quantifying the job variables in a manner satisfactory to the Corps' contracting officer in advance of awarding the contract should go far toward eliminating successful claims, arbitration and litigation.

Dredge Education Programs – Both the Corps and industry personnel could profit by exposure to a coordinated training program expounding the fundamentals of hydraulic dredging, along with estimating techniques based upon normal and improved contract documents. Outdated practices should be replaced with newer proven techniques on which both Corps and industry could agree. A Corps educational program similar to that provided for dredge inspectors, but adapted to contract administrators and estimators, would be appropriate, and hopefully could be made available to all Corps districts.

Conclusions

There is a large potential reduction in dredging claims, arbitration and litigation: (1) by improving soil sampling and reporting techniques; (2) by improving estimating accuracy on the part of the Corps and the bidder; (3) by identifying and quantifying the job variables in the bid; and (4) by making available to the Corps and industry personnel appropriate training programs.

Recommendations

The Corps, the largest potential gainer from the improved program, should finance a small task force effort to outline and agree upon the guidelines for a new program. Then a committee from WEDA, consisting of at least two Corps people, two dredge industry people, one Corps contracting officer, one Corps lawyer, and two dredge consultants could review, modify and put the stamp of approval on the program for the industry.

Biodata

Thomas M. Turner

Mr. Turner received his B.S. in Engineering (Honor Graduate) from North Carolina State University in 1943. He served three years in engineering assignments, afloat and ashore, in the U.S. Navy Reserve. He was employed some 32 years with industry and from 1978 to present is President of Turner Consulting, Inc., a dredge consultant, in Sarasota, Florida.

Mr. Turner is the holder of numerous patents and developer of the Seven Basic Laws of Dredging. He is a lecturer at the Texas A&M Dredging Short Course held annually. He has been consultant to major dredging companies, including Panama Canal Committee and Army Corps of Engineers. He was instrumental in the development of bucket-wheel, production meter, submerged pumps, compensated dredges, etc.

Mr. Turner is a member of the World Dredging Association and is a registered Professional Engineer. He has written numerous articles for the World Dredging Association and is author of **Fundamentals of Hydraulic Dredging** published by Cornell Maritime Press in 1984.

The Requirements and Application for the Use of Simulation Techniques at CAORF as an Engineering Design Tool in the Ship Navigation, Channel Design and Maintenance Optimization Process

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General Introduction

This paper will describe the navigation process as a system and the utilization of simulation as a design tool in the channel design process. The requirements for simulation capability to meet design objectives will be discussed. The need for various data sources such as ship model test, hydraulic current prediction models and field data will also be described.

The information will include:

- Specification of objectives of harbor and waterway design studies
- Functional simulation capabilities to meet objectives
- Research methodology
- Performance indices
- Simulation techniques
- Data base development

This paper will conclude with examples of various simulation project results to explain how the process is applied.

In recent years port design studies have been done on a variety of simulators that vary a great deal in fidelity and sophistication. In addition, there has been research to support the use of low-fidelity simulators for such studies. A number of port design studies capability experiments are reported in the Proceedings of the Third International Marine Simulator Conference (MARSIM '84). This paper will consider the simulator capability needed for port design studies.

Conditions in a Harbor or Waterway

The design of harbors and waterways has increased in difficulty with the number of complexity of factors that must be considered. The demands for safety and productivity often conflict. Increases in the size, speed and specialization of ships have made obsolete all rules of thumb previously used to determine the waterway volume needed to provide an acceptable safety margin. At the same time, increases in costs have made it necessary to minimize dredging. Both hazardous cargoes carried in too-small channels and excessive dredging may present risks to the environment. Harbor and waterway designers need effective tools to assist in their decisions and to justify these decisions to other interests.

Research conducted in the area of navigation of ships in narrow waterways was for many years focused on hydraulic channel testing and simulation of ships' hydrodynamic response in analog or digital computer models. These methods were used to evaluate a single transit of a channel by a ship. Typically, autopilot, rudder and propulsion control algorithms were used to control the model or the simulation. The advantages of such research methods were replicability, and the ability to isolate the study of unique hydrodynamic responses. These research methods provided valuable data about the vessel's physical response in the waterway. The extent to which these vessels could safely transit the waterway, however, could not be ascertained, since these methods failed to account for the variability the pilot and helmsmen introduced.

Recognizing this deficiency during the past decade, several research institutions around the world have integrated the human element into research through the use of ship simulators. By considering the variability man's performance adds to the piloting process, we are truly considering the ultimate safety of the vessel in the waterway; for a waterway can be said to be safe to the extent that under simulated conditions, various ship tracks can be continued within the boundaries of the waterway. Individual differences introduced by pilot and helmsman can be found in both the perceptual/cognitive domain as well as behavior relating to preference and ability. Therefore, measurement of the human factor must encompass a sample large enough to represent the variability of the entire population. The facility appropriate to simulate the man-in-the-loop is the full-scale ship simulator.

Simulation is presented here as such a tool. However, simulation is effective as a tool and the results can be generalized to the harbor only to the extent that conditions in that harbor are analyzed, specified and represented in the simulation. The waterway configuration, ship characteristics, operational practices and the visible surroundings are conditions that will be emphasized in the following sections.

Objectives of Simulation Research for Port Development

The ship simulator can provide operational data which can be used to determine dimension requirements for (a) maneuvering areas of port terminals, and (b) access channels to ports. In the latter case we might be concerned with total channel width, or the width of a special deep cut lane within the channel to accommodate those vessels with greater draft restrictions. Once the dimension requirements have been established, they may serve as a basis for dredging to maximize cost-effectiveness.

A systematic approach is needed to specify dimension requirements as a function of multiple variables such as environmental conditions and ship type. Some of the objectives relating to these variables are stated below:

- Can the full range of existing and proposed user vessels be accommodated in current port facilities?
- Can a proposed dredging plan accommodate all existing and proposed user vessels?
- Which proposed terminal sites are cost-effective in terms of maneuvering area requirements?
- What environmental conditions prohibit safe operations in port waterways?
- What are the aids to navigation requirements of berthing/unberthing and channel transits?
- How do the interactions of dredging configuration, ship type, environmental conditions and aids to navigation modify the dimension requirements?

Of course, how the research results are implemented in designing a particular port should be determined using the criteria of cost-effectiveness within acceptable limits of safety.

The Navigational Process as a System

The navigational process operates as a system composed of the ship, the shiphandler and the environment. The process is diagrammed simply in Figure 1 to illustrate the interdependence of the components. The ship is controlled by the shiphandler (assisted by the helmsman). The environment, including the waterway configuration, contributes the forces that act on the ship and the information available to the shiphandler. For an accurate understanding of how the system will perform in the specific waterway under specific conditions and for an accurate simulation, all the components must be considered and specified. It is the operation of the navigational process as a system that gives the preference to "man-in-the-loop" simulation as it is described in the following sections. Other descriptions of the navigation process appear in the marine simulation literature (Puglisi, 1985; Atkins and Bertsche, 1980; H-10, 1975).

In harbor and waterway design, the focus is generally on changing the waterway's configuration to improve performance of the system, that is, to increase productivity or safety. Because the navigational process operates as a system, performance can also be improved by changing other components of the system. For example, safety can be improved by restricting high-risk ships to daytime, high-slack tide, or one-way traffic. Vessel traffic systems may be useful in some situations. Tugs can be added when the waterway configuration does not allow large ships to move under their own power. Improvements in ship maneuverability, including thrusters, are a very long-term solution. Specialized training for the shiphandler may improve the performance of the system. Improvements to the aids to navigation systems or all-weather navigations systems may be helpful. Unfortunately, in the real world, components of the system, other than the waterway configuration, are controlled by interests other than those of the waterway designer, and there are political and economic barriers that could prevent change. Simulator evaluation of a variety of possibilities may be a tool in the political process. The possibility of trade-offs among components of the system and the use of simulator data to evaluate those trade-offs are discussed in the Coast Guard's Aids to Navigation Systems Design Manual (Smith et al., 1985).

Simulation as a Design Tool

Simulation assists the design and decision process by allowing alternative harbor and waterway designs

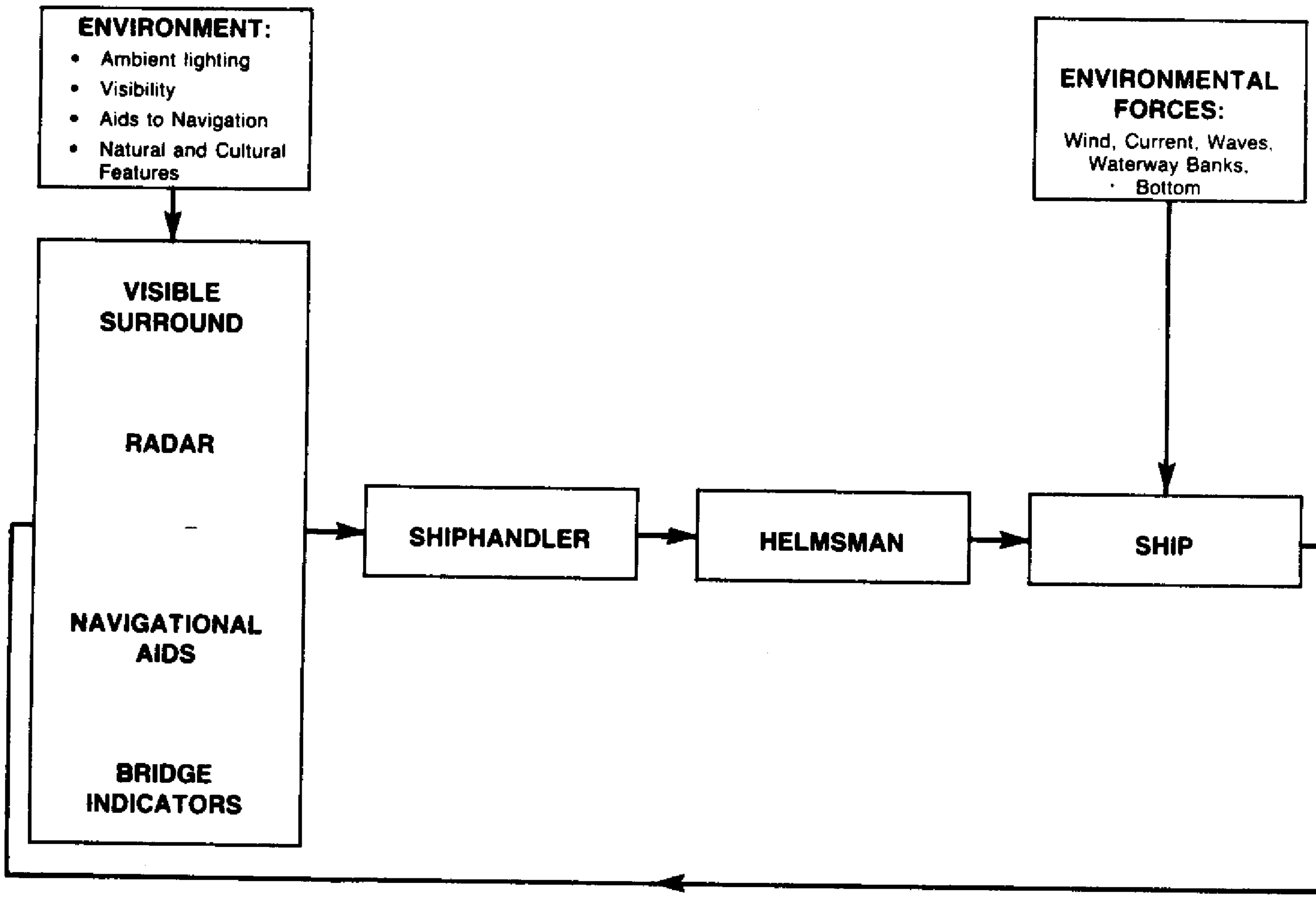


Figure 1. The Navigational Process as a System

to be examined for safety and productivity without real risk and without the far greater investment of implementation of a non-optimized design. By allowing the examination of a variety of alternative designs, together with unique combinations of ship, environmental conditions and operational practices, simulation can provide a design that is the unique solution of conditions in a specific port.

The primary contribution of simulation is quantitative performance data for the design and operational alternatives being considered. A number of methods furnish data for the design process: physical models, mathematical models and man-in-the-loop simulation. "Man-in-the-loop" simulation provides data that includes the entire navigational system, including the variability of the shiphandler. It also allows interested parties to examine proposed designs before they are selected or implemented. Such examination both provides for subjective evaluations to augment the quantitative data collection, and encourages acceptance of a proposed design by those who have the opportunity for early familiarity with it.

Study Factors

A complete examination of the marine transportation system requires the incorporation of all relevant factors and their interactions into the study, including, for example:

- Channel geometry impact on vessel/ship controllability
- Vessel hydrodynamics and aerodynamics and course keeping
- Prevailing currents and wind forces
- Formal aids to navigation, such as ranges and buoys
- Informal land-based navigation aids
- Visibility, ambient lighting and meteorological environmental effects
- Human operator control and decision processes
- Availability and use of vessel maneuvering assistance such as types of tugs, bow thrusters, fenders, etc.

The categories of data required for a port study include:

- Waterway configuration
  - Channel widths and depths
  - Turn types and angles
  - Bank and shoal locations
  - Type and location of hazards
- Environmental Statistics
  - Wind direction and velocity
  - Current direction and velocity
  - Visibility range
  - Unique current conditions
- Aids to Navigation System
  - Types of aids
  - Characteristics and patterns (day and night)
  - Location of aids
- Operational Policies and Conditions
  - Traffic rules and congestion
  - Tug availability and sizes
  - Limits on operations
  - Types of vessels accommodated

Need for Simulation

The need to consider the complex interrelationships among the above factors in order to maintain high validity when examining the marine transportation system tempts one to study problems utilizing the real world (natural environment) as a vehicle. The costs and risks associated with conducting marine transportation research in the field are prohibitive. It would be difficult, if not impossible, to exercise the required scientific controls (i.e., to control extraneous factors) or to collect enough data (i.e., over a wide range of environmental conditions) to form reliable conclusions and recommendations. The most serious drawback is that it would not be possible to evaluate the benefits of proposed changes to marine transportation systems prior to their implementation. For example, the determination of the effects on vessel controllability and safety of a channel deepening project would have to await project completion. By that time millions, perhaps hundreds of millions of dollars would have been expended. These types of problems have in the past been minimized by conservatively over-engineering channel designs. That is, design more channel than optimal to provide adequate safety in the face of unforeseen factors. Such an approach is not cost-effective when considering dredging costs and requirements. It would clearly be desirable to examine proposed modifications to the marine transportation system prior to implementation, be they changes in channel design, vessel design, aids to navigation, operational procedures or any other mitigating factor.

Simulation Research Tools

With these considerations in mind, the Maritime Administration's Office of Research and Development (MARAD) concluded that a shore-based facility was desirable. This resulted in the construction of the Computer Aided Operations Research Facility (CAORF) (see Figure 2). A full-mission real-time simulator piloted by experienced mariners under realistic conditions seemed to have the proper balance between realism, flexibility and cost-effectiveness; however, other models including fast-time simulation are also utilized at CAORF. Since 1976, CAORF has played a significant role in the growth and development of a number of ports, harbors and waterways in the United States and abroad. The analytical tools used by the CAORF staff have included:

- Formal risk management procedures
  - Risk assessment
  - Risk mitigation
- Fast-time computer simulation
  - Hydrodynamically valid ship models
  - Realistic environmental conditions
  - Shallow water/bank/passing ship effects
  - Ship control algorithms

- Ability to represent specific port, channel and operating conditions
- Real-time simulation (CAORF Simulator)
  - Realistic simulation of the total environment
  - Man-in-the-loop performance
  - Assessment of the effect of the human element.

A number of simulation methods are listed in Table 1, along their strengths and limitations. They differ in the degree to which they are inclusive of the entire navigational process, in the time and cost required, and in their requirements for validation. Other comparisons of possible methods appear in the Proceedings of the Sixth CAORF Symposium (D'Amico, 1985b), and the Proceedings of a National Research Council Panel (Crane, 1980).

The first method listed in Table 1 is a physical model of the waterway. A physical model provides a good representation of bank and bottom irregularities and effects on hydraulic forces. Data collected with such a model substitutes for data collected at sea and allows the investigation of alternative configurations that do not exist at sea. Such data is a source for mathematical models of the waterway. The method has high cost and time requirements. While it is a thorough investigation on one critical component, it is not an investigation of the navigation process in the subject waterway. While there may be a question of scaling effects, there is not the question of validity that exists for mathematical models of the waterway. (Physical models of the ship have similar strengths and limitations. They are not listed separately here because they are a component of a waterway design study, not a possible simulation method in such a context).

Fast-time mathematical models allow the combination of the waterway and ship models and such operational factors as transit speed and meeting traffic. To the extent that the waterway and ship models are already available, this method allows for the rapid, inexpensive screening of large numbers of waterway configuration and operational possibilities. Fast-time mathematical models differ from physical models in that the waterway and ship components should be validated separately, and possibly together, for confidence in the ability of the results to transfer to sea. Two types of fast-time simulation that differ in their control algorithms

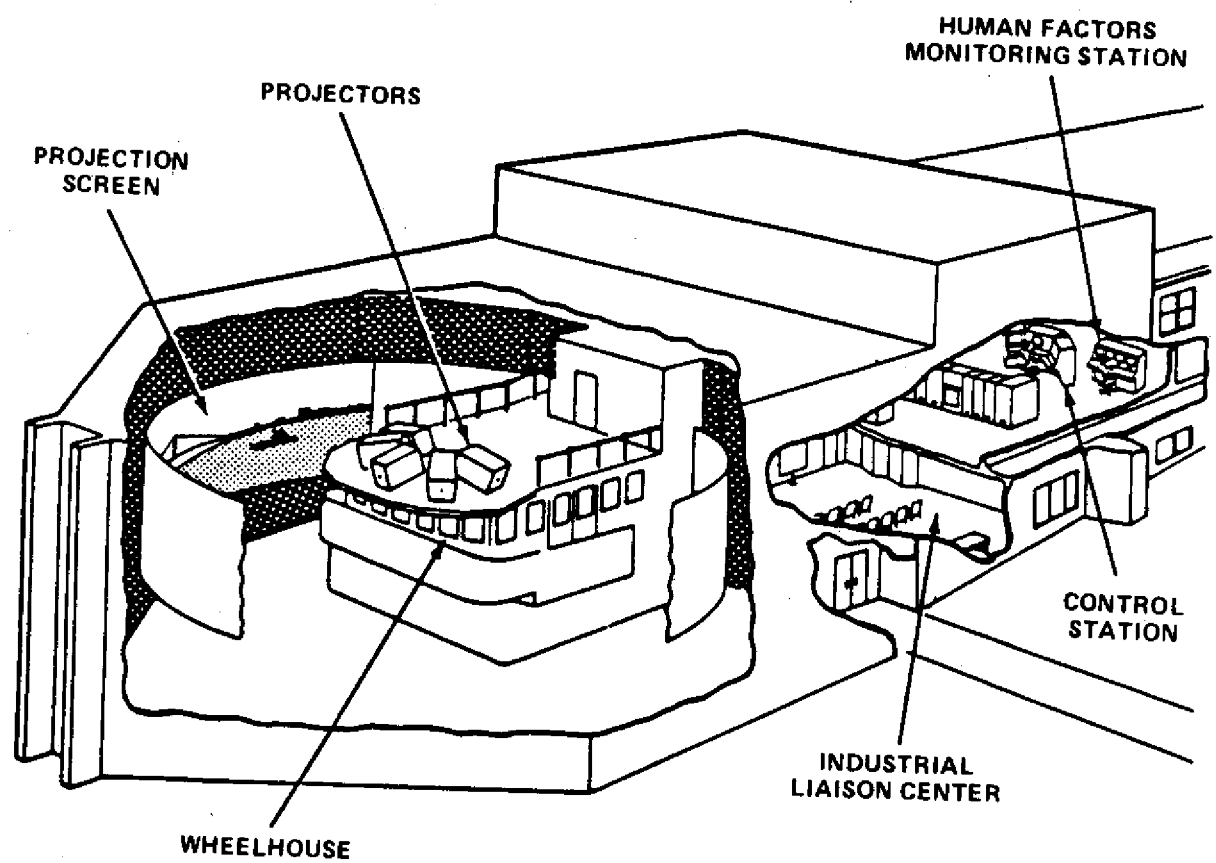


Figure 2. Cutaway of CAORF Building

are listed in Table 1. A direct control model has a predefined series of control orders that are applied as a function of time or distance traveled. The series is established for each waterway alternative that is to be examined. Such a model produces ideal performance of the combination of waterway and operational conditions. The shiphandler and his effect on the navigation process is omitted. An autopilot model recognizes and corrects deviations from the desired track during the transit and, thus, includes some representation of the shiphandler. To prepare the autopilot, shiphandler data must be collected at sea or on a man-in-the-loop simulator, modeled and validated. This effort increases the time and cost beyond that of a direct control model. Examples of the development of an autopilot (Hwang, 1985) and its validation for the Panama Canal Study (Schryver, 1985) have recently been published by CAORF. An autopilot model demonstrates possible performance for the entire system, but does not include the variability that the shiphandler contributes to the system.

The last two entries listed are both real-time man-in-the-loop simulation. For such methods, all components of the navigation process are represented by mathematical models or physical mockups, except the shiphandler. The shiphandler, the most difficult component to model or simulate, is real. Typically, he is a representative of the population of pilots who control ships in the harbor/waterway being investigated. Real-time man-in-the-loop simulation is most inclusive of the navigational process. Progress through a transit is evaluated by the shiphandler, control orders are generated and progress is reassessed, all in real time. Replication of transits allows inclusion of the variability introduced by the shiphandler. Replications of transits by different shiphandlers allow inclusion of the wider variability likely to be encountered at sea. The inclusion of the shiphandler puts special demands on the simulation to provide an interface between the shiphandler and the rest of the system. The two entries in Table 1 differ in the fidelity of this interface. A lower-fidelity interface (for example, a graphic display on a computer terminal and keyboard controls) is obviously less expensive to provide than a high-fidelity interface (For example, a complex view of the visual surround and a ship's bridge mockup). Because it is relatively inexpensive, it allows for the screening of large numbers of alternative waterway designs and conditions. However, the low-fidelity interface raises serious questions about the validity of the shiphandler's behavior, i.e., the extent to which it is representative of such behavior at sea. Because mathematical models of the waterway and ship are used, real-time simulation makes the same demands on the validity of these components as does fast-time simulation.

Research Validity Requirements

In the development of a study relating to engineering design and evaluation (or any other simulation research for that matter), it is essential that the validity of the research be established. The findings from a study would be worthless if its validity were compromised. Two important areas of validity need to be considered. First, the simulation models must validly represent the real world being depicted. Second, the research design must validly address the objectives of the study. That is, a research plan must provide the data necessary to provide a basis for evaluating study objectives. These two lines of validity are briefly described below.

Simulation model validity is generally established through the comparison of the simulation models against data generated through some combination of the following methods. (The examples below are presented within the context of ship model validation but the same logic applies to all simulation models including visual scene, currents, etc.)

(1) Real-World Data – a comparison with real-world measurements or data such as ship test trials.

(2) Model Testing – a comparison with the results from test conducted using measurements derived from scale model tank testing.

(3) Theoretical Estimation – a comparison with performance estimates derived through mathematical extrapolation or interpolation using accepted theoretical models.

(4) Expert Opinion – a comparison with the performance expected and evaluated by experts on the system the simulation has been designed to model.

While most attention is directed towards simulation model validity, which is a prerequisite to a sound research plan, it is equally important to establish a research design having strong validity. Indeed, simulation model validity is a necessary but not sufficient condition for valid research. Once an accurate tool has been developed (the simulation models) it must still be used properly to adequately address study objectives.

Table 1. Methods of Simulation for Harbor and Waterway Design

Method	Strengths	Limitations
Physical model of waterway	Good representative of bank and bottom irregularities Source of data for mathematical models	High cost and time requirements Limited part of system
Fast-time mathematical model: direct control	Relatively fast and inexpensive Screens large numbers of alternatives	Not fast or inexpensive if waterway data must be collected and modeled
Fast-time mathematical autopilot	Relatively fast and inexpensive Screens larger numbers of alternatives Some inclusion of ship-handler function	Not fast or inexpensive if waterway data must be collected and modeled Not fast or inexpensive if shiphandler data must be collected and modeled Omits consideration of shiphandler and information available to him Question of validity of mathematical models No variability to shiphandler function Limited to scenarios for which devel oped Question of validity of mathematical models
Real-time man-in-the-loop simulation: limited display and controls	Relatively fast and inexpensive Screens large numbers of alternatives Includes variability of human operator	Not fast or inexpensive if waterway data must be collected and modeled Question of validity of components other than shiphandler Question of validity of shiphandler performance in response to display
Real-time man-in-the-loop simulation: high-fidelity visual scene, indicators and controls	Most inclusive of components of system Includes variability of human operators Allows demonstration, quantitative analysis and subjective evaluations of proposed designs	Relatively high in time and cost requirement Question of validity components other than shiphandler

Research design validity can be evaluated along four dimensions:

(1) Internal validity – the arrangement of experimental factors and experimental control procedures in such a way that differences observed between engineering designs (or experimental conditions) can convincingly be attributed to differences between the experimental factors and not extraneous variables. For example, if two channel width designs were being compared but the aids to navigation were different for each, then one could not be sure whether differences in vessel performance between the two designs was due to channel width, aids to navigation, both factors, or some interaction between the factors.

(2) Construct Validity – the identification of appropriate measures of the constructs (type of performance domain of interest as identified in the study objectives or hypotheses) under investigation. For example, if the construct of interest is safety, then the measurement of vessel rudder angle alone would represent weak construct validity, i.e., by itself it inadequately addresses the construct of interest.

(3) Statistical Conclusion Validity – the assurance that the comparisons being made between engineering designs are sensitive enough to observe differences between them if differences would exist in the real world. Study elements such as sample size and operational definition of study factors is important in this regard. (The issue is directly linked to the statistical power of the analysis procedures used by this relationship will not be elaborated here).

(4) External Validity – the procedures incorporated into a study which permit the generalization of the results obtained from the sample which was observed to the population which is of interest. Questions of generalization can be directed to pilots, vessels, conditions examined, scenarios and any other element in the simulation which is sampled from the real world. This form of validity is often ignored but is critical to valid research. The performance observed in the study is only a sample. Results and conclusions must then be generalized to the real world. External validity is especially important to engineering design evaluations where the study's findings may be implemented in the real world.

The establishment of both model and research design validity is required of simulation projects. The latter, while often ignored, is of special importance in the evaluation of simulation research capabilities.

Simulator Fidelity and Validity

The preceding discussion introduced the concepts of "fidelity" and "validity." Both affect the confidence to be had in the degree to which the simulation can represent the real world. Fidelity is the simpler concept. Here, it is used to mean the inclusion in the simulation of real-world elements. Sometimes the elements are relatively major; for example, the out-the-window visual scene or the presence of current in the waterway. Sometimes the elements are relatively minor; for example, the presence of a water tower or some local variation in the current. Validity is the more complex concept and the more difficult to achieve and demonstrate. Here, it refers to the ability of the simulated system or some major component to operate as it would at sea.

There is a relationship between fidelity and validity. Fidelity, the inclusion of the real-world elements, contributes to validity. The omission of real-world elements may make valid system performance on the simulator impossible. It is not appropriate to consider the provision of high fidelity a substitute for validation of the system. However, there is generally more confidence in a high-fidelity system than a low-fidelity one. As an example, if no difference is found between waterway design alternatives on a low-fidelity simulator, there is considerably more uncertainty as to whether there would be no difference between the alternatives at sea or whether the simulator did not allow the difference to be observed. It is appropriate to say that the lower the fidelity of the simulation, the greater the uncertainty about its validity, and the greater the burden to demonstrate validity.

Fidelity adds to the validity of simulation; it may add considerably to the the cost. Therefore, it is necessary to examine exactly what fidelity is adding to the simulation's effectiveness. As an example, the inclusion of a water tower may contribute to "face validity," the immediate impression of realism, and therefore to user acceptance of the simulator or the waterway design. However, if the water tower is not used by the local pilots as an aid of opportunity, its inclusion will contribute little to "empirical validity," or measured system performance. It is the latter that provides the quantitative performance data for waterway design decisions. Because a high-fidelity simulation can be quite costly, the demonstration of validity and user acceptance for low-fidelity simulation can mean future savings in cost.

The most frequently used approach to validation is to concentrate on a single major component or subsystem. The favored subsystem is the ship model because of its centrality to the simulation process, fast-time or real-time. Validation frequently consists of comparing ship tracks or other ship status measures produced on the simulator to those produced at sea. Recent samples of such validation include CAORF's validation of its ship models for the Panama Study (CAORF Staff, 1984). Other subsystems should be validated. CAORF has had ongoing studies to validate its major subsystems and has provided examples to literature. Early in its history, it published a validation study of its basic ship model, its visual scene, and the behavior of shiphandlers on its bridge (CAORF Staff, 1979). The results of continued validation of these varied components were reported in the Proceedings of MARSIM '78 (Pollack, 1978) and MARSIM '81 (McIlroy, 1981).

There is no commonly agreed upon methodology for validation, no criteria for what constitutes successful validity, and no point to which validation of a simulator is complete. What is necessary in validation is that the simulation be examined and that all involved parties – the researchers, the shiphandlers and the harbor/waterway designers – agree that validity is sufficient to give them confidence in the results. A review of validation studies or methods is listed in the following references (Puglisi et al., 1985; Smith et al., 1984; D'Amico, 1984).

Working rules on the fidelity and validity of simulation follow:

• The more critical a subsystem or special effect is to a particular study, the more important is its fidelity/validity in the simulation.

• The newer or more unusual a special effect is for a simulator, the more important is its validation.

• The lower the fidelity of a simulator or subsystem, the more important is its validation.

Outline of a Simulator-Based Harbor/Waterway Design Study

The section establishes the relationships between a simulator-based harbor/waterway design study and the simulator capability needed. A study is described as a number of steps, each of which makes demands on simulator capability. Here, simulator capability is broadly defined as including the research capabilities that surround the simulator.

General Project Elements

A research methodology involves the coordination of many project elements. A general schematic of a typical sequence of project elements is presented in Figure 3. Beginning with a **Statement of Work**, which provides a general presentation of the engineering problems and consideration related to the project, a project team is assembled from the CAORF staff which includes the appropriate areas of expertise required to define the project in greater detail. That team usually includes a research methodologist, a marine transportation specialist, an engineer and a computer scientist. An expert in some special technical area may be included if needed. The project team must then determine the **Project Specifications**. That is, the specific engineering designs and related problems and issues. Included in this effort is a determination of the hypotheses to be tested, problems to be solved, potential candidate solutions to the problems, important variables which will be incorporated into the study, and many other details which define the simulation project. Specifying the project involves a great deal of data collection and interaction between the project team and other groups such as the project sponsor, i.e., U.S. Army Corps of Engineers, U.S. Coast Guard, local Port Authority and local pilots.

When the important details of the project have been specified, the simulation enters the development phase. This is when the simulation project is translated from relatively abstract terms into a concrete investigation, i.e., operationalizing the study. Initially, three parallel activities occur: preparing the Presimulation Report, developing the ownship model, and developing the models of the waterway (see Figure 3). The **Presimulation Report** provides the study background, objectives, research methodology, simulation modeling requirements, participants (such as local pilots) required, and all details necessary to conduct the study. Each project element required to conduct the study must be specified in detail. A list of these project elements is provided in Table 2. This document is reviewed by the project's sponsor and relevant interested parties. This ensures that the study has been defined as required. If **Vessel Model Development** is required it is accomplished at this time.

The mathematical models of the waterway and the ship and their interaction are critical to a design study. The necessary models, or approximate versions, may already be available. An accurate simulation of the waterway may require extensive data collection at sea or preliminary physical modeling. The ship model may require new data from full-scale trials or from scale models. New data has to be incorporated in the models. Model development may be done by the simulator's own staff or by other facilities. The development of these components may constitute a substantial part of the study's effort and cost.

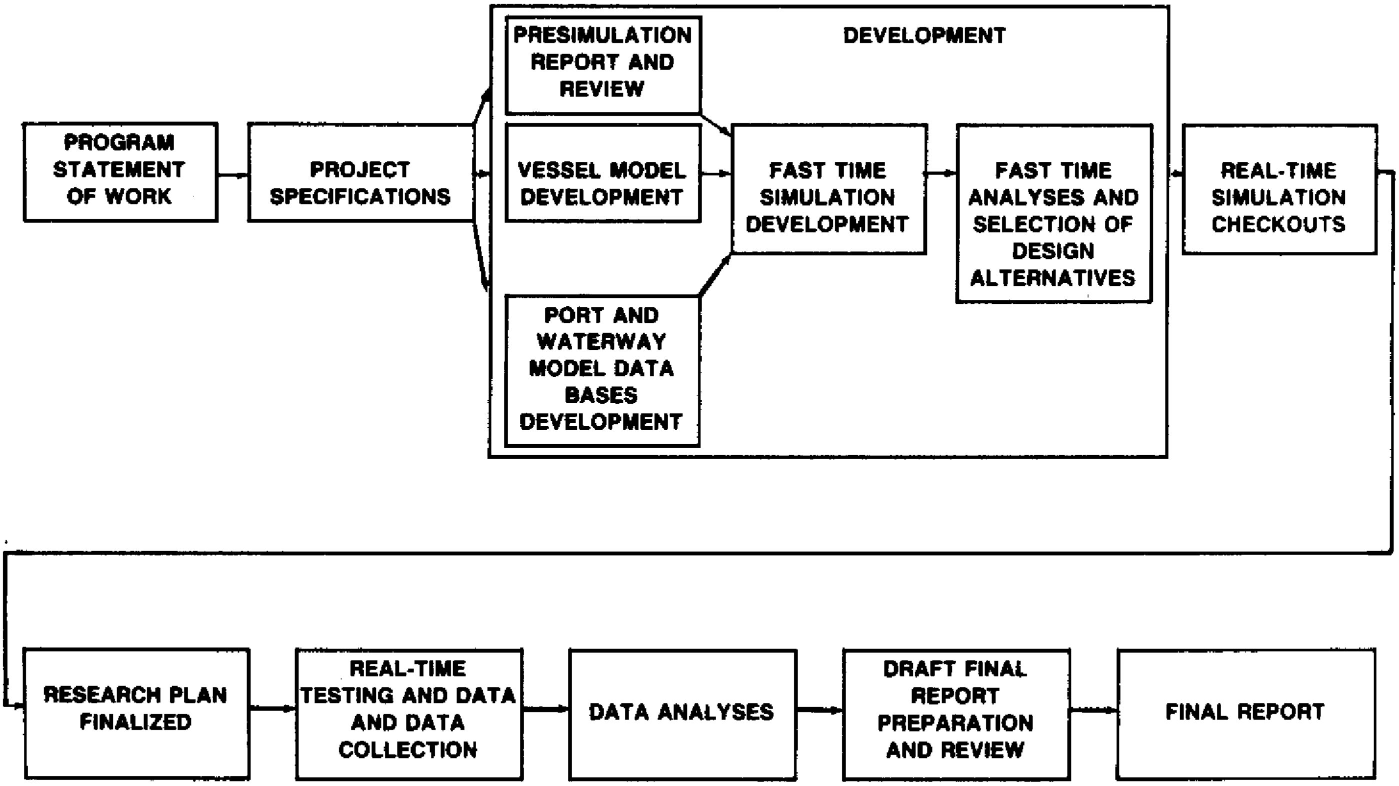


Figure 3. A General Schematic View of the Sequence of Project Elements

Table 2. Research Design Plan Elements

Formation of Study Objectives
Identification of Study Factors
Definition of Study Factors
Experimental Factor Organization
Test Participant Requirements
Scenario Construction
Testing Program Procedures
Measurement Requirements
Data Analysis Strategies

The **Port and Waterway Simulation Model Data Bases** are developed also. Each waterway, e.g., the Hampton Roads channels, requires several "data bases" to provide local mariners with a realistic representation. Man-in-the-loop simulation is representative of at-sea performance to the extent that the simulation allows and encourages the shiphandler to perform as he would at sea. If he generally uses the view-out-the-window as his main source of information at sea, the most valid representation of his performance and the resulting system performance will be obtained when adequate information is provided by the simulator. The preliminary analysis can include the navigational chart, the light list, the photographs taken from the ship's bridge, and interviews with local pilots. An adequate visual scene is the most difficult to achieve and the most costly component of a simulation. It is discussed further in this paper and in a separate CAORF document (Williams, 1985). If low-visibility conditions are of special interest, the local radar conditions or any all-weather navigational systems typically used should be analyzed for inclusion in the simulation.

A very elaborate example of the initial preparation for simulation is the Panama Canal project recently completed at CAORF (D'Amico, 1985; Puglisi et al., 1984). In addition to using its own expertise, CAORF contracted with the Swedish State Ship Experimental Institute (SSPA) and the Stevens Institute of Technology to support the development of a valid mathematical model of a Panama ship. The Panama Canal Commission (PCC) contributed comprehensive real-world data of ship transits through the canal. These data were used for validation of the ship models used in the simulation (CAORF staff, 1984).

Some studies include **Fast-Time Simulation** in the development phase. This simulation utilizes the models developed for the real-time simulation (where mariners control the vessel) but the vessel is controlled by an autopilot. The autopilot is a mathematical model of the human pilot. Since it is run by computer, the autopilot is not constrained by real-time information processing limitations of a human pilot. Nor is the simulation constrained by the real-time maneuvering restrictions of an actual vessel. Hence, many simulated transits can be made in a very short time – and without lunch breaks, rest periods, etc. The fast-time simulations are generally used as a screening tool. That is, to screen out poor engineering designs prior to simulating in real-time with real mariners. In this way only the best designs are tested in real-time which is a cost-effective method of conducting research. The autopilot, however, is only an approximation of a real-time testing. Real-time testing is still needed and is, in fact, the real test of an engineering design. The fast-time simulation has other uses as well, such as quantifying the impact of critical study variables on vessel maneuvering and sensitivity testing (testing the sensitivity of vessel responsiveness within a range of error estimated to be inherent within some input data such as current predictions).

When the development phase is completed, the study enters **Checkouts** where a pilot from the area being modeled is brought to CAORF and transits the waterway, pilots the vessel, and is exposed to all experimental conditions that will be examined in the actual study. This is the time when any "bugs" in the modeling or the research design are worked out. Following the checkout, the **Research Plan Is Finalized** and the **Testing Program** is conducted where all study data is collected. When all testing is completed, the **Data Analyses** are conducted.

The methods of data analysis are critical both to the formulation of conclusions to be drawn from the study as well as the study's validity as discussed in the previous section. Table 3 provides an overview of the data analytic approaches employed in most CAORF investigations. There are three levels of analyses. The first, track plots, provides a graphic "bird's-eye" representation of either a single transit through a waterway or a composite of many transits thereby providing information about the total area used in a channel by all vessels under a defined set of conditions. The next level of analyses, descriptive statistics, provides statistical descriptions of study factors and experimental conditions (combinations of study factors) in terms such as central tendency, e.g., arithmetic average, and variability, e.g. standard deviation. These descriptive statistics summarize important measures of performance. The final procedures for generalizing beyond the sample observed to the population of interest. Parameter estimation techniques allow generalization from sample statistics (such as average or standard deviation) to the corresponding population parameters. Significance testing procedures permit a determination of whether the observed differences between experimental conditions represent real differences or are due to chance (random) variation or noise in the study. All these statistical analysis procedures are important in drawing conclusions regarding engineering design proposals.

Table 3. Data Analysis Procedures

Track Plots
- Individual Transits
- Composite Envelopes

Descriptive Statistics
- Study Factors
- Experimental Conditions

Inferential Statistics
- Parameter Estimation
- Statistical Significance

When data analyses are completed and evaluated, the **Draft Final Report** is prepared. This report is reviewed by the sponsor and when all reviews have been received, the **Final Report** is issued.

Design Alternative – A Research Design Illustration

The comparison of design alternatives places special demands on simulator capability. The factors that will change from alternative to alternative must be represented with a high degree of fidelity and validity. Changed conditions should be demonstrably and accurately different from the baseline conditions. Alternatives to be evaluated may be dictated by local conditions, selected by the harbor/waterway designers, or recommended by simulator research staff. Generally, in harbor/waterway configurations, for example, these are depth, width or return radius. They may include transient environmental conditions such as current flow or wave height, or they may involve operational changes or changes to the aids to navigation system or to all-weather navigational systems. The factors considered for change will determine which simulator subsystems will be critical.

A concrete example of research design is presented in Figure 4. This design was developed to evaluate channel width alternatives for their adequacy in maintaining vessel safety in a meeting and passing situation. The design employs both fast-time and real-time simulations as part of the testing program. As shown in the figure, the design can be characterized by four phases. The logical assumption made for this study was that the new (alternative) channel was to be no less safe than the presently existing channel.

The first phase will utilize fast-time simulation to screen alternative designs to select the most promising alternative. The specific steps by which this will be accomplished are shown in the figure. In the next phase the existing channel condition would be modeled and transited by local pilots to provide baseline safety levels. The third phase will involve the modeling of the selected alternative. Pilots will transit this channel and safety levels will be determined. In the final phase, the safety of the two designs – existing and alternative – will be compared. If the alternative provides safety levels at least as good as the existing design, it will be recommended as adequate.

One final issue is the method by which safety is determined. Safety is a complex construct which does not lend itself to univariate (single variable) measurement. Instead, safety is approximated from the simultaneous

Table 4. Performance Measures to Be Used In the Analysis of Relative Safety of Alternative Channel Design Modifications

Vessel's Proximity to Channel Bounds Measure
- Frequency of channel exits
- Average distance from channel boundary
- Farthest distance away from channel centerline
- Variability of distance from channel boundary

Vessel's Proximity to Traffic Vessel
- Closest point of approach (CPA)
- Average distance of ship from traffic vessel
- Variability of ship's distance from traffic vessel

Vessel Controllability Measures

Yawing Characteristics
- Variability of heading
- Average absolute rate of return (yaw rate)
- Variability of rate of return

Swept Path
- Average "swept path" during bridge passage
- Variability of "swept path"

Rudder Activity
- Average absolute rudder angle
- Variability of rudder angle
- Number of rudder reversals

Assist Tug
- Number and deployment of tugs

Pilot's Evaluations of Conditions

Pilotage Evaluation Rating Scale
(The pilot's ratings of his transits)
- Cognitive Load Scale Score
- Stress Scale Score
- Task Difficulty Scale Score
- Shiphandling Scale Score
- Pilot Workload Estimation Score
- Composite Workload Score

Pilot Opinion Questionnaire
- Various open-ended questions pertaining to the experimental conditions

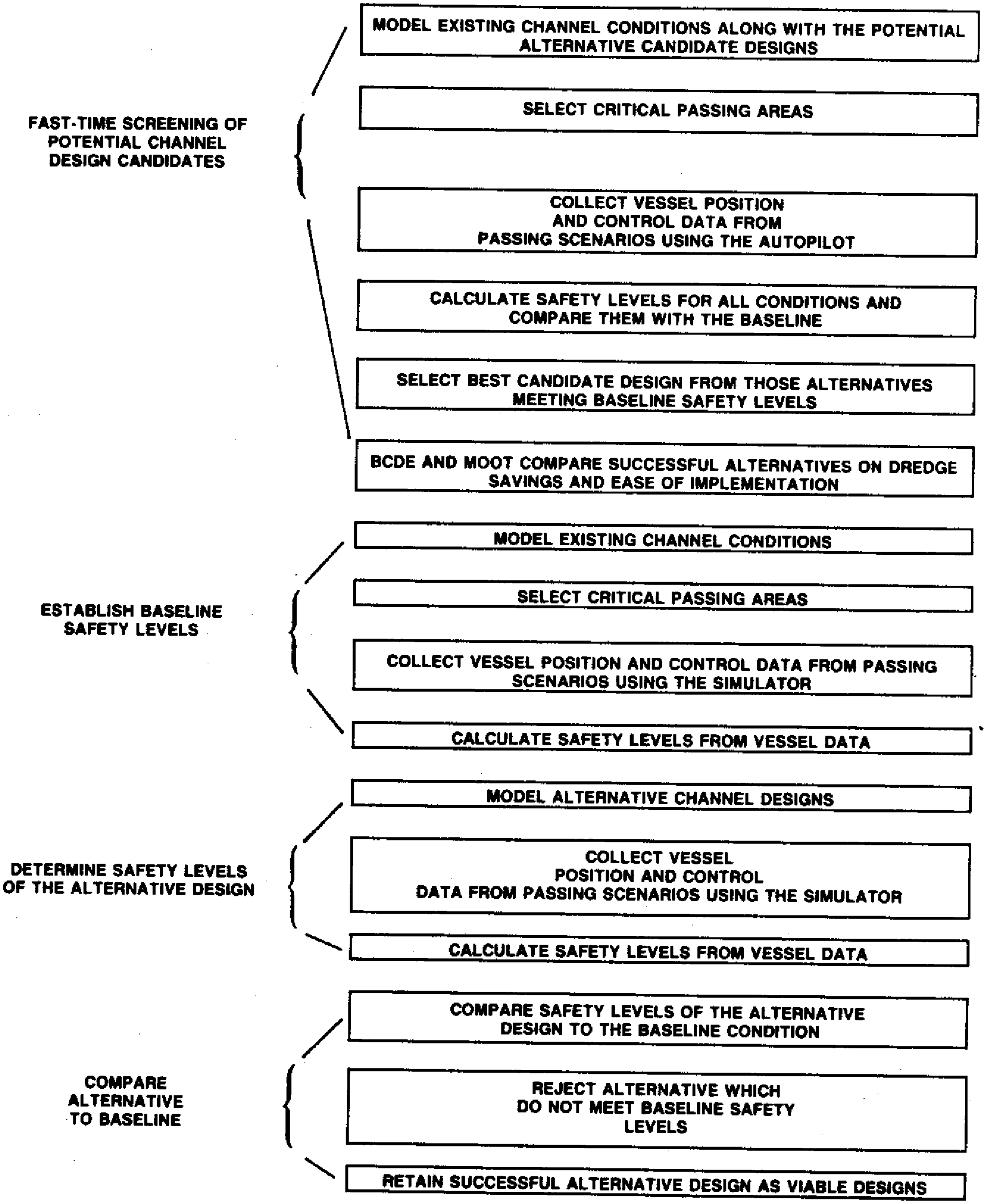


Figure 4. Methodological Approach to the Evaluation of Alternative Channel Designs for Passing Vessels

consideration of many measurement dimensions. Table 4 depicts the range of measures required to adequately address safety of vessel transits in the study described above. The logic of this approach is inherent in man-machine systems technology. Such a systems approach characterizes the vessel transit event. Three dimensions of system performance are evaluated and several measures are used to assess each. The first is vessel proximity to other structures, such as channel bounds, passing ship and other structures in the waterway. Maintaining adequate proximity from other structures is a central objective of the system. Next, some amount of vessel control surface activity is required to achieve adequate vessel positioning. Vessel yawing behavior, swept path, rudder activity and assist tug requirements (along with other variables) describe this activity. An engineering design would not be considered safe if excessive vessel force were required. Finally, a human operator must make control decisions and execute these along with other tasks. One would not want to exceed his capacity or make the task of controlling the vessel exceedingly difficult. Hence, measures of operator workload, stress and reported difficulty, as well as opinions, are obtained.

All three dimensions of measurement are required to adequately assess the safety of navigation designs. The Panama Canal study recently completed at CAORF illustrates the use of fast-time simulation to select alternatives for real-time simulation. Fast-time simulation was used in the earliest design stage to examine all reasonable channel layouts, under all expected operational conditions, for each of eight curves. The matrix resulted in over a thousand alternatives. Preparation for the fast-time screening required the collection of real-time man-in-the-loop data on CAORF's main bridge and the modeling of the pilot's correction process for an autopilot (Schryver, 1985; Hwang, 1985). It also required the development of new computer techniques to direct the screening with a minimum of operator direction. Only a very few alternatives will be evaluated with a full-scale, man-in-the-loop simulation.

Experimental Design

Recent examples of the use of this general experimental design include CAORF's evaluation of alternative arrangements of the channel passing under the Sunshine Skyway Bridge done for the Port of Tampa (O'Hara, 1984). The Panama Canal Study is designed in this way. The Panama Canal Commission has accepted as a standard to safety the present meeting of two 85-foot beam ships and plans to reconfigure the channel to allow two 106-foot beam ships to meet with comparable safety margins (D'Amico, 1985c; Puglisi et al., 1984). The simulation study has defined the dimension of that reconfiguration.

Conclusions and Recommendations Based on the Design Study

The value of a simulator-based harbor/waterway design study is in the usefulness of the findings to their ultimate user, the harbor/waterway designer. As is the case throughout the study process, a multidisciplinary research team, consisting of individuals trained and experienced in harbor/waterway operations, research methods and simulation, is necessary for results that will generalize to sea. A multidisciplinary staff can interpret simulator-based findings in a form most appropriate for the designer.

Simulator Capability Recommended for Harbor/Waterway Design Studies

This section recommends simulator capabilities for harbor/waterway design studies. It provides a functional description of what should be available to the human factors researcher or to the shiphandler-in-the-loop. It does not describe the simulator software or hardware design that will provide the capability.

The assumption developed previously, that the real-world conditions that are of interest in a specific harbor/waterway determine the capability needed to study them, is continued here. Here, this assumption means that even the use of a high-fidelity simulator will sometimes require the development of new features for a specific study. At other times, capabilities available as a facility may be unnecessary or may provide only noncritical embellishments for a specific study.

Simulator capability is divided into subsystems for convenience in description. Any division into subsystems will be arbitrary. Usually, description of a specific simulator is divided into subsystems to correspond to its software/hardware design. Here, no specific simulator is intended. The primary division is between (1) capabilities that are common to fast-time mathematical model simulation and real-time man-in-the-loop simulation and (2) capabilities that are unique to real-time man-in-the-loop simulation. The emphasis is on the latter, as was the case in previous discussions.

Mathematical Models of the Ship Hydrodynamics, the Waterway Configuration and Environmental Forces

It is critical that models of the ship hydrodynamics, the waterway and environmental forces, and their interactions have high fidelity and validity for harbor/waterway design studies. Such studies demand more of the models than other simulator applications, for example, shiphandler training, ship's bridge design, or aids to navigation systems design.

The division here is into three subsystems: ship, waterway and environmental forces. The ship hydrodynamics model must have the ability to interact with "data bases" representing the other two subsystems. "Waterway configuration" and "environmental forces" are listed as separate subsystems. This division is not because they are implemented separately by a simulator, but because they are unique to a specific harbor/waterway study. They may require the collection of data at sea or from scale models, the mathematical modeling of the data, and the validation of the simulated effects.

The behavior of the vessel will, to great extent, depend on the bottom topography. Fine features of the bottom such as the location of channels, strong localized water currents, and depth variation must be carefully modeled since they will have a large effect on the behavior of the vessel in transiting the harbor. The source of data for this information may come from navigational charts, tidal current charts or, for certain critical features, through direct measurement.

All these features must be accurately modeled in the data base of the simulator. CAORF employs a grid system for identifying gross features of depth and currents over the bottom. Fine features of the bottom topography such as the location of channels and the height of the associated banks, depth variation within the channel and currents are stored in data records on disc and keyed to the location of the ownship.

Figure 5 is a pictorial example of fine structures stored in this data record. The bottom data base is checked out by transiting a ship down to waterway to assure that the banks are in the proper channel depth has been modeled. Current structures are tested by checking that the currents produce the proper force and turning moment on the vessel

Wherever possible these checkouts are conducted using pilots familiar with the vessel and waterway, and comments are solicited from the pilots on the realism of the simulation.

Mathematical Model of Ship Hydrodynamics

A high level of sophistication is necessary. In general practice, three degrees of freedom (surge, sway and yaw) are used for deep-draft ships in relatively sheltered waters (Levine and Puglisi, 1985). A discussion of what constitutes a sophisticated model is beyond the scope of this paper. The level of simulation needed is a matter of recent concern elsewhere (Case et al., 1984; McCallum, 1984; Dallinga et al., 1984).

Port modifications must be planned to satisfy projected user demands over an extended period of time to maximize the cost/benefit of the improvement project. Normally, the problem involves the extension of port operating limits to a family of real and proposed vessel types differing in length, beam-width, draft, freeboard, horsepower and maneuverability. A required component in the study is a mathematical hydrodynamic model for the ship in motion with the proper set of response coefficients for the ship's propulsion and control forces. Mathematical models of ships have progressed to a stage at which there are a number of ship types available as models. Additionally, captive hydraulic model tests can produce good estimates for models' coefficients, given the ship's physical characteristics. Today's mathematical models include factors such as:

- Bank influence
- Shallow water effects
- Tugboat forces
- Pier and dolphin forces
- Anchor forces
- Passing ship effects
- Wind and current effects

In the following sections, as an example, we shall briefly discuss how each of these effects is modeled in CAORF, and indicate the types of supplementary data necessary to provide an accurate simulation of these effects.

Bank Influence

In CAORF a distinction is made between wall forces produced by a single wall and wall forces produced

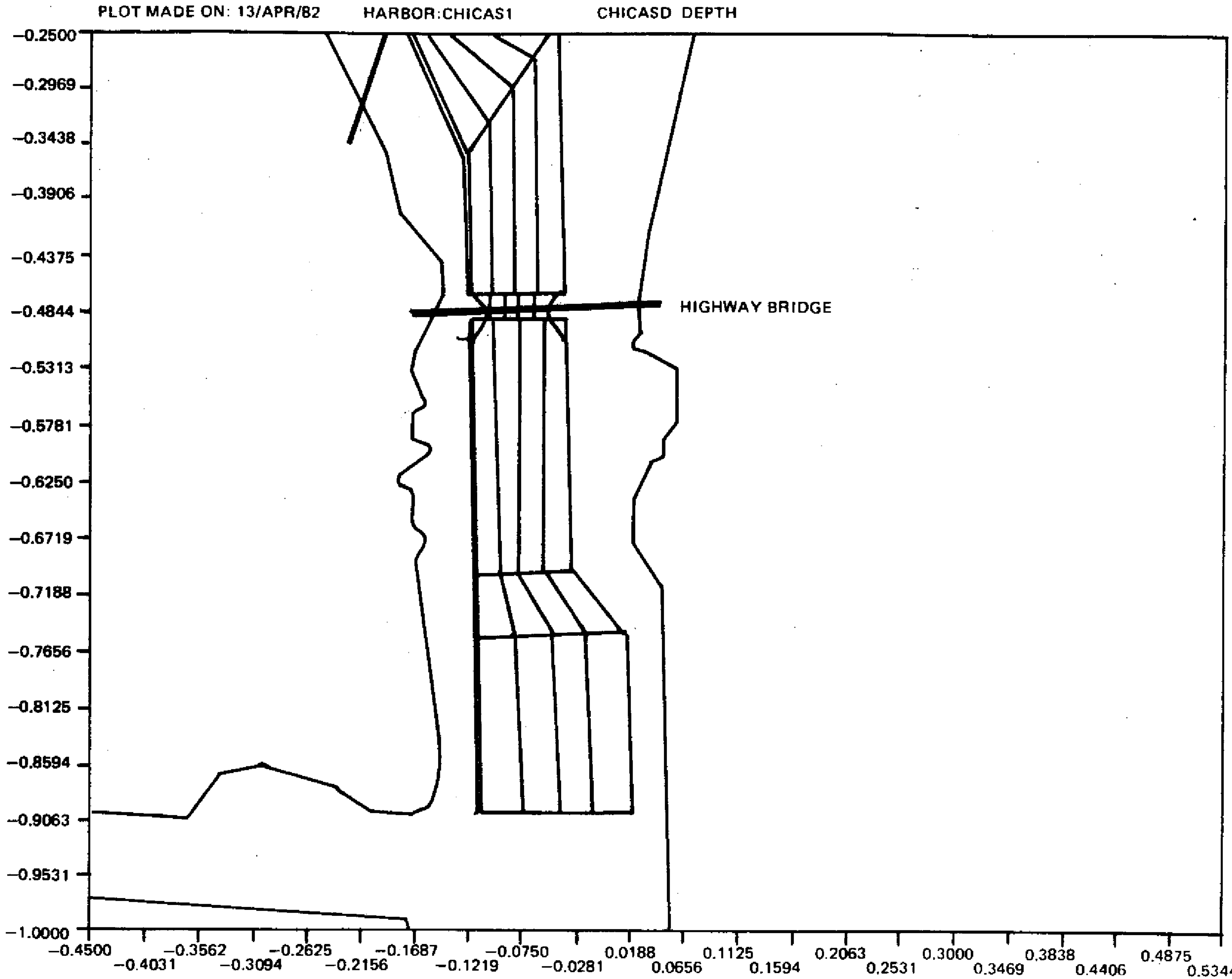


Figure 5A. Depth Data Base (Uniform Depth)

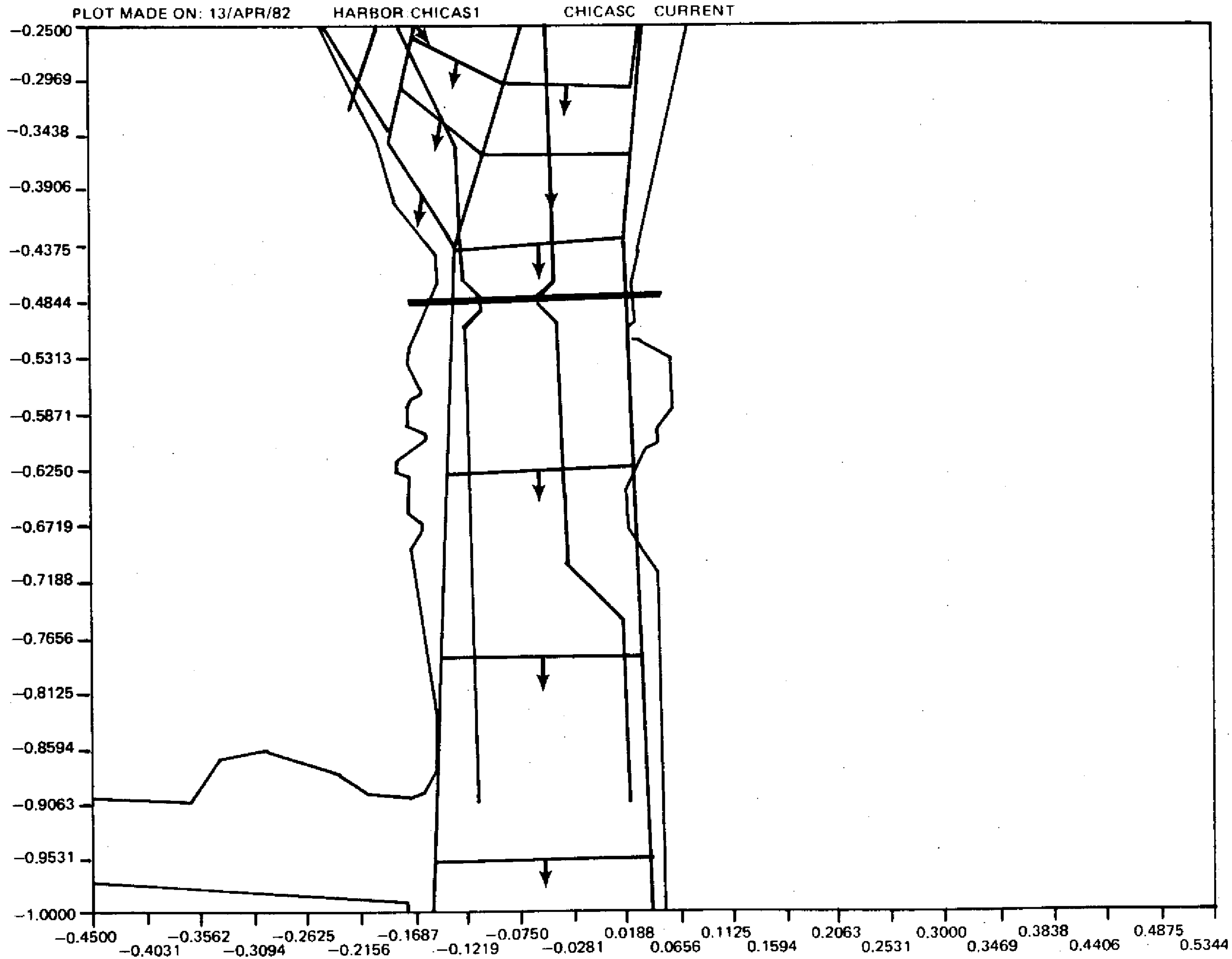


Figure 5B. Current Data Base (Uniform Current)

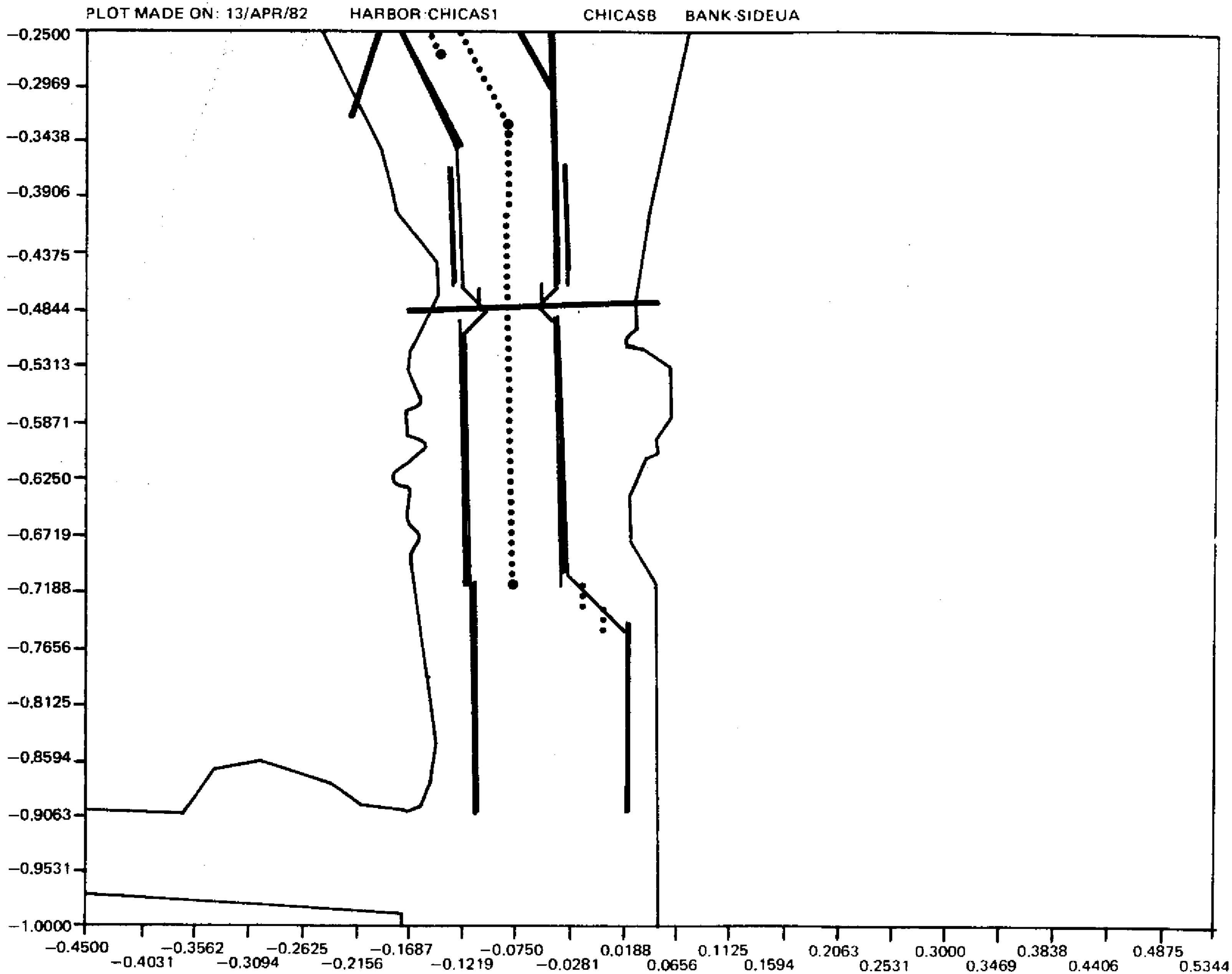


Figure 5C. Channel and Bank Data Base

by the two walls of a channel or canal. The normalized yaw and sway force produced by a single wall is modeled as:

$$N_B{}' + M_{1a}C_{1a}A_{ref1}{}^{f}(Y_{OB})$$

$$Y_B{}' + M_{1b}C_{1b}A_{ref1}{}^{f}(Y_{OB})$$

whereas for channels, the interaction effect of the second wall produces a function of the form:

$$N_c{}' + M_{4a}C_{4a}A_{ref4}Y_c{}'^{+M}{}_{10a}C_{10a}A_{ref10}Y_c{}'^{3}$$

$$Y_c{}' + M_{4b}C_{4b}b_{ref4}Y_c{}'^{+M}{}_{10b}b_{ref10}Y_c{}'^{3}$$

where the terms:

M_{ij} = side wall structure multiplier which is a function of the height of the bank sway velocity and relative heading of the vessel.

C_{ij} = confined water multiplier which is a function of the depth of water, and the channel width to own ship beam ratio.

$Y_C{}'$ = normalized lateral distance measured from the channel centerline.

$f(Y_{OB}{}')$ = a function of the normalized lateral distance measured from the bank line.

The multiplier factors are curve-fitting polynomials which are used to fit model data or theoretically derived data. The bank configuration is implicit in the type of structure used in the model test. The math model is able to compensate for variations of the channel parameters (e.g., the wall height above bottom, water depth in the channel and channel width), so that channels or banks of different combinations of values for these parameters can be modeled.

For experiments where bank influence is a critical factor, model tests should be made for the channel wall configuration of interest and this data used to generate coefficients for the simulator.

Shallow Water

Shallow water has a marked effect on the maneuverability of vessels. The handling characteristics of a vessel rapidly change in shallow water where the depth to draft ratio varies from 1.5 to 1.1. In the CAORF simulator, each hydrodynamic coefficient of the ownship is multiplied by a shallow water multiplier of the form:

$$K_{SW} = (1 + {}_DS_{w1} + {}_D2S_{w2} + {}_DS_{w3})$$

where:

$${}_D = H/D_w - {}_o$$

The shallow water coefficients (Swi) are obtained by modeling the ownship for a number of conditions of water depth so that the functional variation with respect to depth of each hydrodynamic coefficient can be determined. This functional variation is used to generate the shallow water coefficients. It has been shown that for high block coefficient vessels the shallow water variation of the coefficient is very closely the same so that it is not always necessary to perform extensive model testing to develop the shallow water characteristics. Each hydrodynamic coefficient is varied continuously as a function of water depth so that the simulationmodel can operate at anydepth.

Tugboat Forces

In port development it may be necessary for economic and safety reasons to require tug escort and assistance for large vessels transiting the crowded harbors. In addition, tugs will be needed for berthing, deberthing and turning the vessel in crowded waterways.

CAORF has two methods for applying tugboat forces on the ownship. The first method is to apply a simple force at any point on the hull during the simulation. The force amplitude and direction can be manually controlled by dial controls at the control station, and the maximum force is limited to the maximum static bollard pull of the tugboat being modeled. This simulation technique used in many simulators is somewhat limited since the bollard force of the tug is independent of the velocity of the ownship. This becomes most apparent in escorting functions where the tug and ownship may be moving at speeds of several knots.

In the second method the tug hydrodynamics is modeled, and the tug is subjected to wind, current and ownship-tug interaction forces. In this method, the available force that the tug can exert on ownship is related to the velocity of the ownship. Braking forces in excess of the static bollard pull can be obtained when the ownship drags the tug due to the added tug resistance which can be greatly increased by increasing the angle of attack of the tug as it is pulled through the water.

In berthing maneuvers where velocities are very low the two methods give approximately identical results.

An accurate simulation of tug assistance on ownship requires the development of coefficients which characterize the inherent maneuver capability of the specific tug being used. These coefficients can be entered into the tug subroutine.

Pier and Dolphin Forces

Berthing and turning maneuvers can be simulated by using the dolphin subroutine. A pier can be simulated by using several dolphins implated along a straight line. Forces due to currents acting on the ship against the pier can be measured, and the effect of the ship backing off from a pier into a strong tidal current can be simulated.

Dolphins, in combination with tugs or natural forces such as wind or currents, can be used to maneuver large ships in confined waters. In addition, dolphins can be used in experiments to determine placement of dolphins used to protect vessels passing through narrow passage ways.

The dolphin forces are adjustable to simulate train forces and energy-absorbing characteristics of actual structures. Simulation of the energy dissipation characteristic of shock-absorbing fenders can be accomplished by using one set of coefficients as the vessel compresses the dolphin. When this force begins to move the vessel, a second set of coefficients are used to model the force during recoiling of the dolphin. In this manner the effect of an energy-absorbing fender can be simulated (see Figure 6).

Considerable useful data on pneumatic and energy-absorbing can be obtained from manufacturers of these devices.

Anchor Forces

In addition to using the anchor as a mooring device it can also be used to help maneuver the vessel in confined waters. An anchor simulation subroutine has been added to CAORF. This anchor dynamically simulated the four stages in anchoring shown in Figure 7. In normal anchoring operations, if the ship has sufficient backing force the ship will dynamically oscillate between stages 2 and 3 as the ship slowly moves the anchor until the backing force is dissipated.

In some cases the anchor is let out on a short chain to prevent it from embedding, so it can act as a brake to slow the ship down or cause it to turn.

The anchor function uses characteristics of existing anchors which have been tested for holding strength by the Bureau of Ships. Estimates must be made for the water resistance on the chain, and on the added mass of the anchor when it begins to drag.

Passing Ship Effects

The CAORF passing ship program is a six-term Fourier series approximation of the force and moment functions obtained from model test data. These terms are multiplied by a distance multiplier term which is a function of the lateral separation distance of the two ships, a relative angle correction term that accounts for relative ship headings and a velocity multiplier term that accounts for ship velocities other than the reference test velocities.

For experiments where passing ship effects are critical, a model test should be made using scale models of the specific ship involved, to obtain precise force and moment data as a function of separation distance.

Wind and Current Effects

Steady wind or gusting wind can be applied to the ownship dial controls at the control station. Gusting wind is produced by a random generator which varies the wind speed over specified limits about the mean wind speed. The wind direction can also be made to vary over specified limits about the mean.

The ownship aerodynamics is simulated by a five-term Fourier series of the aerodynamic cross section of the ship.

In order to generate aerodynamic coefficients, a broadside and profile scale detail drawing of the ship is required. The loading of the ship must also be specified.

The response of the ownship to uniform water currents is to cause the vessel to drift in the direction of the current. Of more interest is the response of the ship to non-uniform circulating currents.

The ownship detects such current by measuring the current strength at four places along its length to determine water current gradient that cause rotation of the ship. Current gradients are produced by current channels modeled in current data base as mentioned earlier.

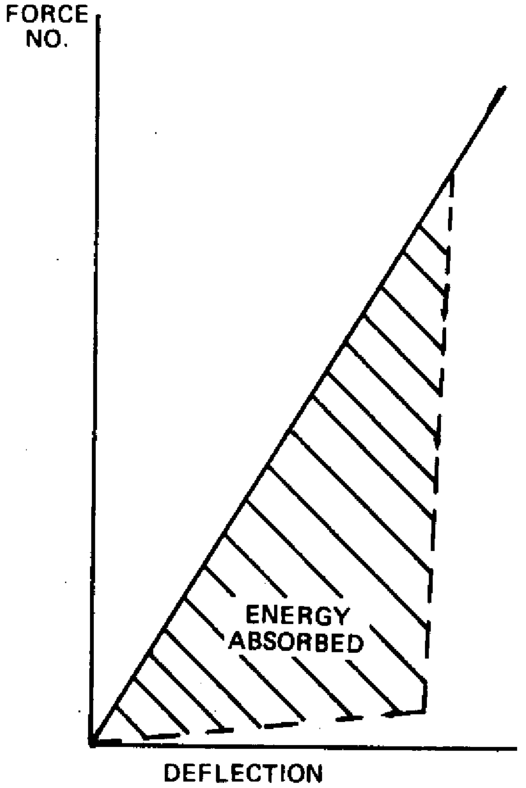


Figure 6. Simulation of Energy-Absorbing Fender

Waterway Configuration Data Base

The model of the waterway configuration is central and critical to a harbor/waterway design study. The general findings of the study will be valid to the extent that critical aspects of the configuration are represented in the simulation. For a specific harbor/waterway some or all of the following have to be represented:

- waterway depth
- bottom contours
- sidewall configuration
- turn configuration
- dolphins
- docks

The necessary size of the "gaming area" will depend on the harbor/waterway to be simulated.

Environmental Forces Data Base

Environmental forces are important and may be critical in any specific harbor/waterway design study. Mathematical models that simulate the forces imparted to a ship by the following environmental factors may be necessary to ensure the fidelity of the simulation:

- current
- wind
- waves

In addition to real-world data, this data is provided by output derived from hydraulic simulation models. The first hydraulic models were scaled physical reproductions of study areas involving steady-state flows. These

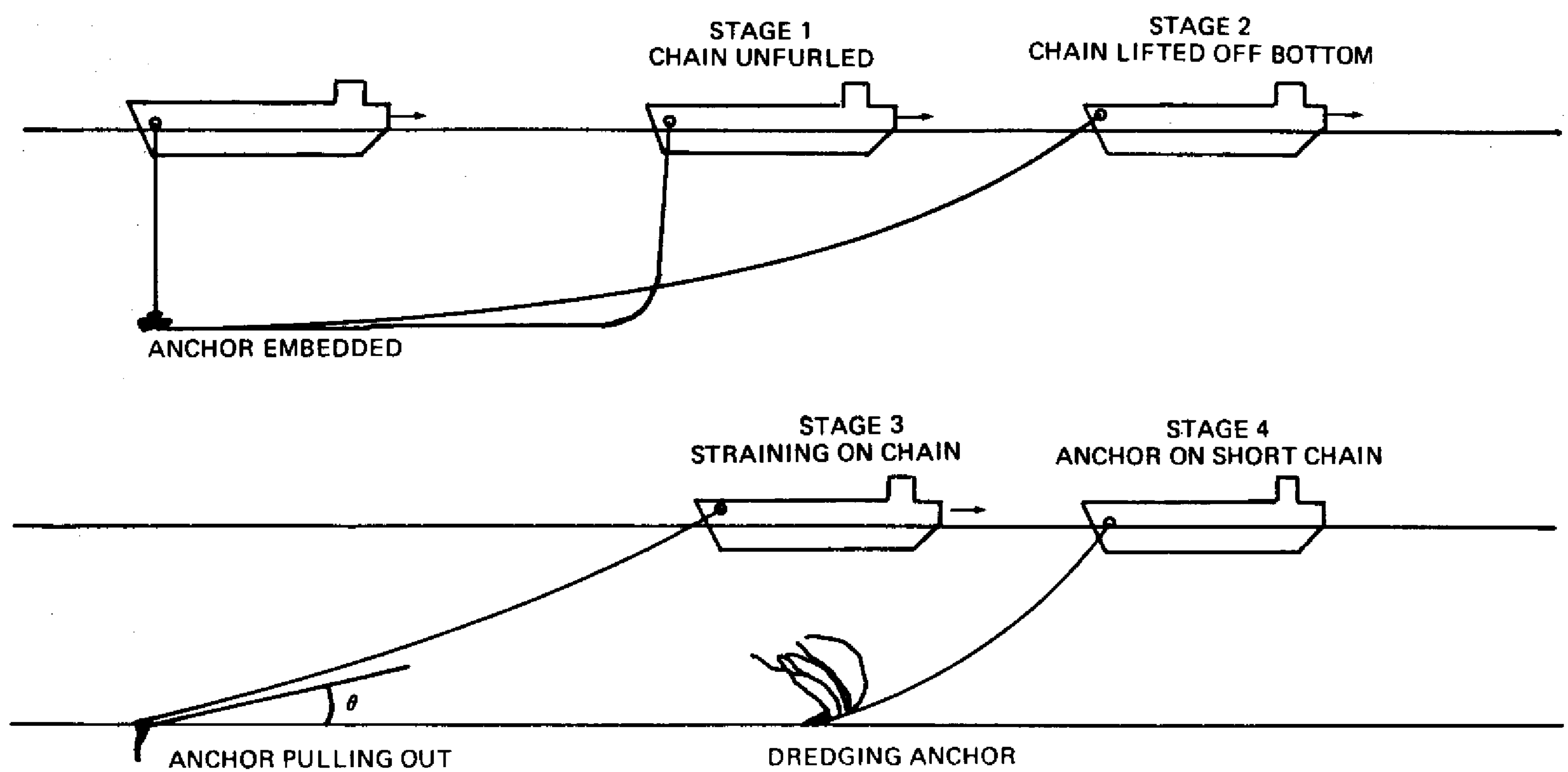


Figure 7. Stages in Anchoring

models became more sophisticated, with the addition of tides, salinities, movable beds and variable flows, vessels and simulated ice. During the 1960s, computer-based numerical models were introduced to study hydraulic phenomena.

The waterway configuration may cause spatially local changes in these events that may need to be included in the simulation. The design problem may include a consideration of operational restrictions on transits; for example, larger ships restricted to move favorable environmental conditions. The consideration of such alternatives requires valid simulation of transit changes in these events.

Physical Models

Physical models are used in three basic methods for channel design. For deep-draft channels, the scaled bathemetry is reproduced and current velocities are measured under various tidal and/or river flow conditions. For shallow-draft channels, scaled remote control vessels are navigated through the study area under a variety of channel and structure alignments and discharge conditions. Towing tank tests are conducted to measure vessel characteristics for inputs to numerical models.

Usually the vastness of area to be modeled for port and harbor studies requires using a distorted scale model. These models, therefore, rarely incorporate vessels into the study. However, they can be used to predict current velocities and directions which are important to vessel operations. Riverine models are often non-distorted and frequently incorporate remote-controlled tow boats to assess navigability into lock approaches and through reaches with bridges. Because these are scaled models, time is scaled also. Response time has to be exaggerated.

Numerical Models

Numerical models have become increasingly popular as a tool to study hydrodynamics of various waterways. They are not hampered by scaling effects.

Math Models can incorporate various water quality parameters that physical models cannot. In addition, they can include wind conditions. Generally, they are much faster to build and calibrate. They do require basically the same data for model adjustment and verification as physical models. Numerical models can also study vessel behavior as a function of channel design and current and wind conditions.

Hybrid Modeling

The hybrid modeling approach integrates physical modeling, numerical modeling and analytical modeling to produce superior results. (W.A. McAnally et al., 1986. The approach to two-dimensional and three-dimensional hybrid modeling).

Incorporating Man-In-the-Loop

The ship's bridge, the visual scene, and the radar are the subsystem required for incorporating the man-in-the-loop. There is no separate discussion here of low-fidelity graphic display. A low-fidelity display presenting a panoramic or out-the-window view can be considered the low end of a continuum of visual scene fidelity. A display presenting a plan, or bird's eye view, has some of the characteristics of a radar display. If a choice is to be made between two possibilities for a graphic display, the panoramic view has been found more effective (Shuffel, 1984; Perdock and Elzinga, 1984).

The bridge, the visual scene, and the radar contribute to an observer's judgment of the "full validity," or realism, of a simulator. A full-size ship's bridge, a high-fidelity visual scene, and a "simulated" real radar set obviously have high face validity. In turn, such a judgment contributes to the observer's acceptance of the simulator, the design study and of the eventual implementation of the findings at sea. They may also contribute indirectly to the validity of objectively measured man-ship environment system performance by affecting the shiphandler's motivation.

These three subsystems are not the only factors that contribute to an observer's or a shiphandler's acceptance of a simulator-based design study. The experienced shiphandler who participates as the man-in-the-loop is in a position to be the most severe critic of a simulation. His sophistication in shiphandling and his "local knowledge" of a harbor/waterway also allow him to appreciate other aspects of the simulation and the design study. Some other factors that influence his acceptance include the following:

- The behavior of the ship, waterway and environmental models. These are very important factors to the experienced shiphandler, especially if the simulation is advertised as representing familiar real-world conditions.

• The degree to which design alternatives are workable and, preferably, are improvements over the familiar baseline conditions.

• The professionalism of the research staff as reflected in their understanding of both simulation and the real-world situation, in the appropriateness of the briefings and simulator familiarization, in their questions and in their responses to his comments.

• The degree to which a lower-fidelity bridge, visual scene and/or radar supports his accustomed control of the ship. As an example, if his favorite water tower is represented only by a square, he may be satisfied if it moves appropriately across the screen during a transit.

Ship's Bridge Mockup, Indicators and Controls

The fidelity of the ship's bridge is made up of many dimensions, including:

• the overall size and construction of the mockup.

• the realism, completeness and arrangement of the equipment.

• the resemblance of the bridge to the ship type represented by the hydrodynamic model and/or visible ship's bow.

• the presence/absence of a motion base.

All these dimensions contribute, separately and in combination, to varying degrees to the realism of a particular design study in the judgment of a particular observer or shiphandler. Descriptions of what constitutes high- and low-fidelity ship's bridges have been done for the Training and Certification Project, Phase I (Hammell et al., 1982). Detailed description of specific high fidelity ship's bridges (Levine et al., 1985; Carpenter, 1983) and low-fidelity bridges (Hanley et al., 1984) are also available elsewhere.

The degree of fidelity of the ship's bridge mockup that is necessary for user acceptance and motivation can only be established subjectively and by the agreement of those involved. A dependence on subjective assessment is probably more appropriate for this subsystem than for any of the others. The acceptability of the bridge should not be assessed in isolation, but in the context of the entire design study.

The ship's indicators and controls can be more critical to the effectiveness of a simulation than the overall fidelity of the bridge. The indicators and controls can vary in the following dimensions:

• their presence/absence on the simulator bridge.

• their degree of physical similarity to real equipment.

• the adequacy of their functioning in support of the ship-man-environment system.

Their presentation should be determined by what is generally found on the design ship and by what the pilots involved generally use. Sample lists of what appears on simulator bridges are presented in Table 5. The fidelity of indicators and controls may vary from completely artificial displays and switches to real equipment. Their fidelity contributes to user acceptance and motivation, as does the overall fidelity of the bridge. The adequacy of their functioning in support of the ship-man-environment system is critical to the validity of the objectively measured performance of the system, generally the primary objectives of a design study. As previously noted, it was suggested that the lower the fidelity of a simulation, the greater the burden to demonstrate validity. At a minimum, artificial indicators and controls should be designed with the involvement of human factors experts and experienced shiphandlers. These experts should have the opportunity to inspect and try out an early model before it is incorporated in a complete simulation. A recent formal evaluation of an individual shiphandling trainer (CAPTAINS) (Moynehan, et al., 1985) found inadequacies in indicators and controls that could have been remedied early in the design process.

Sometimes a novel piece of equipment – for example, a precision navigation display – may be among the design alternatives evaluated for a particular harbor/waterway. In such a case, the fidelity/validity of the simulation of the device is especially critical to the overall validity of system performance. The simulated device may precede the real thing and be part of its design process. The issue is then not of fidelity, but of the ability of the device to support system performance. Presumably, if it is effective on the simulator, it will be effective at sea. Such a design evaluation may occur in the context of a harbor/waterway design study (O'Hara, 1984), but is actually quite a different simulator application (Cooper et al., 1980, 1981).

Visual Scene

The recommendations here for a moderate-fidelity visual scene made with consideration of a number of sources:

• a recent CAORF paper on visual cues (Williams, 1985).

• the U.S. Coast Guard's Aids to Navigation Systems Project's simulator validation (Smith et al., 1984).
• the Naval shiphandling Training report (Hanley et al., 1982).
• the reports from the Training and Certification Project, Phases I and II (Hammel et al., 1980, 1981).

The recommendations are summarized in Table 6.

Color: The visual scene should be in color. Color is essential for aids to navigation and ship lights. It is an important contributor to face validity. It contributes to subjective resolution, perceived brightness and perceived depth (Williams, 1985). Some effect on performance has been found (Hammel et al., 1981). Color is well within the present technology and is probably cost-effective. The number of colors possible should be correlated with the complexity of content possible. (See **Resolution, Content**)

Resolution: As high a resolution as possible is preferable. Resolution is one determiner of the possible complexity of the visual scene. It is also a determiner of the smoothness of perceived motion. In these ways it contributes to face validity and to the perceptual cues necessary for the empirical validity of system performance (Williams, 1985). There are a number of moderate and high-fidelity simulators with resolution of approximately 3 to 4 arc minutes or less. This resolution is within the present technology. (See **Content**)

Table 5. Sample Lists of Bridge Equipment from a High-Fidelity Bridge and a Low-Medium Fidelity Bridge

High Fidelity Bridge: CAORF[1]

CAORF Characteristics as Delivered 1976:

Communication (VHF, HF, Intercom, sound-powered telephone)
Engine failure alarms
Gyro steering, hand steering, NFU steering
Radars (2)
Ship's power failure alarms
Speed log
Throttle and telegraph engine order
Wind speed and direction indicators
Fathometer
Propeller, rpm indicator
Rudder angle indicator
Ship's whistle
Steering failure alarms
Thrusters

CAORF Enhancements:

Fathometer – 8/76
Marine Radar Interrogation System (MRITS) – 6/82
ROTI – 12/78
Radar and collision avoidance equipment – 9/82
Medium speed diesel engine sounds – 7/82
Loran C – 7/76
Pelorus – 12/78
Doppler and Dual Axis Sonar – 6/82
Tugboat console – 2/82

Low-to-Medium Fidelity Bridge:

Ship Controls:
- A ship's wheel and helm unit
- An engine order telegraph

Ship's Indicators
- Two gyro repeaters, one on the steering stand and one mounted with an azimuth circle
- A staff rpm indicator
- A rudder angle indicator
- A ship's clock modified to show scenario time

Radar PPI
- A 16-inch PPI simulating a generic 3 cm radar

Navigation Display Unit
- The navigation display unit processes a variety of information displays

Notes:
1. Taken from Levine et al., 1984.
2. Taken from Puglisi et al., 1984.

Table 6. Characteristics Recommended for a Moderate-Fidelity Visual Scene

Color:	– minimum, 4 steps of 3 primary colors (red, green, blue) – more if CONTENT complex
Resolution:	– minimum, approximately 3 to 4 arc minutes both horizontally and vertically – higher for complex CONTENT
Scene Update:	– 30 times per second
Field of View:	– horizontal minimum, 120 degrees – preferable: wider lateral view and rear view – vertical, 20-30 degrees
Content:	– day and night if operationally relevant – minimum: bow image; simple aids, simple objects, simple stationary traffic ships – preferable: bow image, complex aids, complex objects, moving traffic ships, bridge, landmass – ideal: detail and texture

Scene Update: The scene update rate determines the smoothness of the presentation of motion and the accuracy of its perception (Williams, 1985). A number of moderate- and high-fidelity simulators have update rates of 30 times a second and is within the existing technology.

Field of View: The field of view necessary for a specific harbor/waterway design study depends on the layout that is to be investigated. The Training and Certification Project, Phase II, has demonstrated the relationship between specific scenario content and the needed horizontal field of view (Hammell et al., 1981). A minimum of 120 degrees, +60 degrees to each side of longitudinal axis of the ship is recommended for transits in narrow channels. The field of view allows close objects – buoys, traffic ships – to be observed as they pass along the ship's bow to the bridge wings. A field of view of 240 degrees allows observation for passing abeam and overtaking situations. A still larger field of view is desirable for a design study that requires the observation of an open harbor. A rear view allows the shiphandler to observe pairs of buoys falling away behind the ship and to use rear ranges and may be required on rare occasions. Both the horizontal view abeam and the vertical view from the eyepoint determine the possibility of observing close-in tugs and docks.

Content: If the harbor/waterway conditions include both day and night transits, the simulation should allow for both. Nighttime conditions are more difficult at sea. Nighttime conditions are less demanding of a simulator. It is tempting to rationalize that a nighttime-only simulation will result in a safe design for the difficult nighttime conditions and day will be no problem. This logic minimizes risk, but it does not minimize cost. It does not allow the harbor/waterway designer the option of trading off operational restrictions for dredging; for example, restricting larger ships or two-way transits to daytime.

A minimum image content includes a bow image, simple aids and other objects, and a stationary traffic ship. There is evidence that the bow image contributes to the shiphandler's perception of the harbor/waterway by providing a reference from which to judge distance and motion (Moynehan et al., 1982; Bertsche et al., 1981). A bow image – for example, of a containership – may block a view of close aids, etc. (Kaufman, 1984). The bow image of the typical ship should be included.

Simple aids and objects allow a minimum representation of a harbor/waterway. The simulator validation study done for the USCG's Aids to Navigation Systems, Design and Evaluation Project, demonstrated the effectiveness and the limitations of a simplified visual scene (Smith et al., 1984). The study compared ship tracks taken at sea with those taken in an aids-only visual simulation of the channel. For that portion of the channel where the real-world channel was "aids only," the tracks matched. In one section of the channel where a turn at sea was sparsely marked by aids but surrounded by close landmass, the tracks were very different. There, the aids-only simulation could not support the same system performance. The at-sea conditions to be simulated determine the simulator capability needed.

The capability for a more complex visual scene allows the presentation of a meeting traffic ship, which may be essential in harbor/waterway design, additional traffic ships, tugs, docks, bridges, cultural objects and landmasses. Such an increase in the fidelity of the visual scene is a powerful contributor to the face validity of the entire simulator and to the range of scenarios that can be meaningfully evaluated.

The complexity, detail and texture of the visual scene are important contributors to the perception of depth and motion (Hochberg, 1978). These factors are critical to the fidelity of the visual scene and the validity of man-in-the-loop system performance (William, 1985; Williams and D'Amico, 1980).

Radar: High fidelity radar simulation can be an intrinsic requirement of various harbor/waterway design studies.

• A novel use of radar or related equipment (for example, RACONS) might be a design alternative. In such a case, a high level of fidelity and validity is demanded of the radar simulation.

• The conditions to be evaluated are low-visibility conditions during which the shiphandlers at sea make extensive use of radar. In such cases radar is required. If the use of radar is constant across all the alternatives evaluated, its fidelity/validity is less critical.

• For adequate visibility conditions the use of radar should not be automatic. Two factors need to be considered. First, at-sea shiphandlers may use radar to examine a large area for the presence and location of traffic, for the alignment of buoys, etc. However, they may depend on the view-out-the-window for moment-to-moment navigation. In the latter case, radar will contribute to face validity, but it is not necessary for valid system performance (empirical validity).

Project Illustrations

The approaches and considerations that have been mentioned to have been utilized at the CAORF facility will be described with examples.

Panama Canal Commission Research

In the effort to increase the throughput of large vessel traffic in the Panama Canal, the Panama Canal Commission (PCC) initiated a study of canal modification necessary to permit two-way traffic of Panamax-size vessels throughout is length. At present, the Gaillard Cut is the narrowest section of the Canal (see Figure 8). It is 500 feet wide with several curves, making the meeting of Panamax vessel operationally hazardous. In order to increase the Canal's future throughput, the Gaillard Cut would have to be modified to accommodate large vessels in meeting situations.

The PCC is considering the redesign of the Gaillard Cut to achieve safe meetings between Panamax-size vessels while incurring the least excavation and maintenance costs. The objective of the Widening Study is to determine the specific dimensions which will afford a reasonable balance between excavation costs and safety. Technical, operational, economic, financial and environmental considerations are being evaluated to provide the information necessary for the PCC Board of Directors to render a decision regarding the project.

To gain assistance in the required technical analyses, the PCC entered into a cooperative, interagency arrangement with MARAD early in 1983 to allow for the utilization of CAORF in the evaluation of various channel configurations. The PCC's decision to utilize CAORF followed a worldwide evaluation of simulation facilities and research capabilities. CAORF offered both the fast-time and real-time simulation capabilities necessary for the determination of an optimum navigational channel.

The Panama Canal Widening Study is one of the largest single endeavors in CAORF's history. It required the expansion of CAORF's in-house fast-time simulation analysis capabilities, the development of new, reliable and valid measures of safety in passing situations, and the exercise of virtually all of the real-time simulation capabilities of CAORF. In addition, the development of data for a valid mathematical model of a Panamax ship, so important to the validity of the entire study, was undertaken by the Swedish State Ship Experimental Institute (SSPA) and Stevens Institute of Technology under subcontract to MARAD. SSPA performed hydraulic model tests of the Panamax vessel to provide test data to characterize the inherent ship maneuverability in deep and shallow waters, and the ship-channel interaction effects of meeting ships in straight channels of 650-foot to 750-foot widths. Bank interaction was also tested in straight sections and a bend. The series of tests was one of the most extensive ever undertaken in shallow and restricted waters in varied conditions. The resultant mathematical model of the Panamax-size vessel was used by CAORF as the design vessel which PCC pilots conned past another Panamax vessel in the newly designed channel. The "validation" vessel is a Series 60 Class vessel and is the largest vessel that is currently permitted to pass another vessel of the same size in the Gaillard Cut. The development of a model of the "validation" vessel was

PROFILE OF PANAMA CANAL

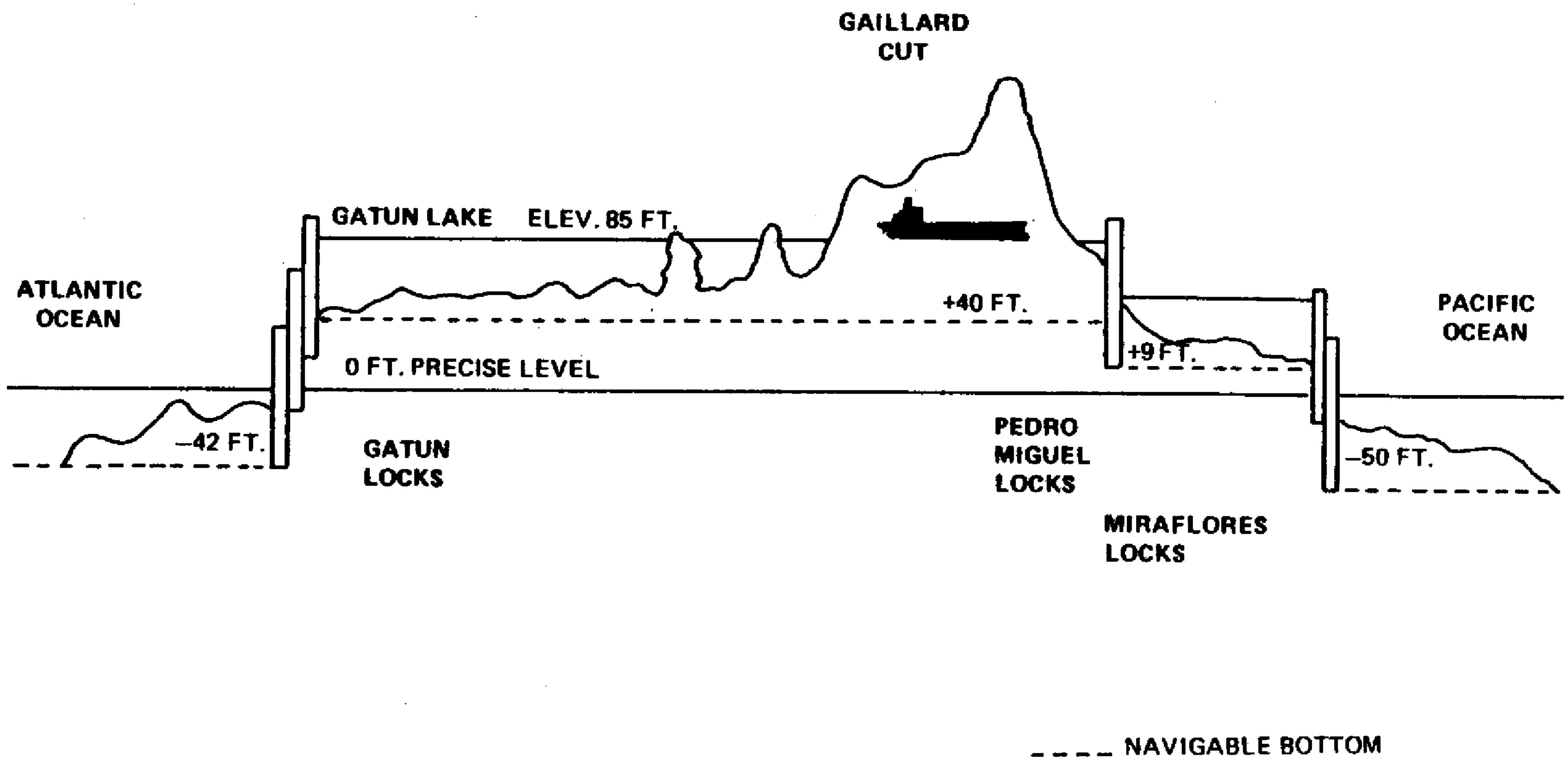


Figure 8. Panama Canal Profile

accomplished by Stevens Institute of Technology, and was based upon real world data, Maritime Administration model data and theoretical considerations.

Real-world data, upon which the development of the "validation" vessel is partially based was collected in July 1983 as a collaborative effort between CAORF and PCC. Ship positions in the Canal were recorded by photographing the radar plan position indicator presentation at 0.5-minute intervals. Aerial photographs of the ship were taken by helicopter at 0.5-minute intervals. Ship positions were also recorded by using land-based reference transponders. Pilot commands and the helmsman's responses were recorded on the audio portion of one of the video cameras located near the helmsman's stand.

Continuous monitoring of the wind-speed and direction, propeller RPM, ship speed, achieved helm and gyro compass heading was also accomplished. During the occurrence of a meeting situation, the relative velocity of the passing ship was measured using a doppler velocimeter. The distance to the passing ship was measured using a tripod-mounted range finder.

The real-world data collected at the Panama Canal in July 1983 was the primary data against which the performance of the CAORF Model was compared. Additional data has also been collected to generate one of the largest existing sets of validation data for ship performance in restricted waters. Details of the methodology and progress to date can be found in the Sixth CAORF Symposium Proceedings, Kings Point, NY, May 1985, and the 19th Dredging Seminar, Western Dredging Association Annual Meeting, Baltimore, MD, October 1986.

CAORF, U.S. Army Corps of Engineers (USACE) and Port Authority Cooperative Research

The safety and productivity of a port is ultimately dependent on human performance during vessel transit and berthing operations. The ability of a pilot or master to safely navigate a ship depends on the complex interaction between human performance, harbor design parameters, environmental conditions and ship response characteristics. Ship operations simulation of the required comprehensiveness can provide the required capability to quantify vessel performance under human control to address a broad spectrum of questions relating to harbor and waterway development. The various districts of the USACE and local port authorities have utilized simulation in the design and analysis process.

USACE – Design and Analysis Approach and Applications

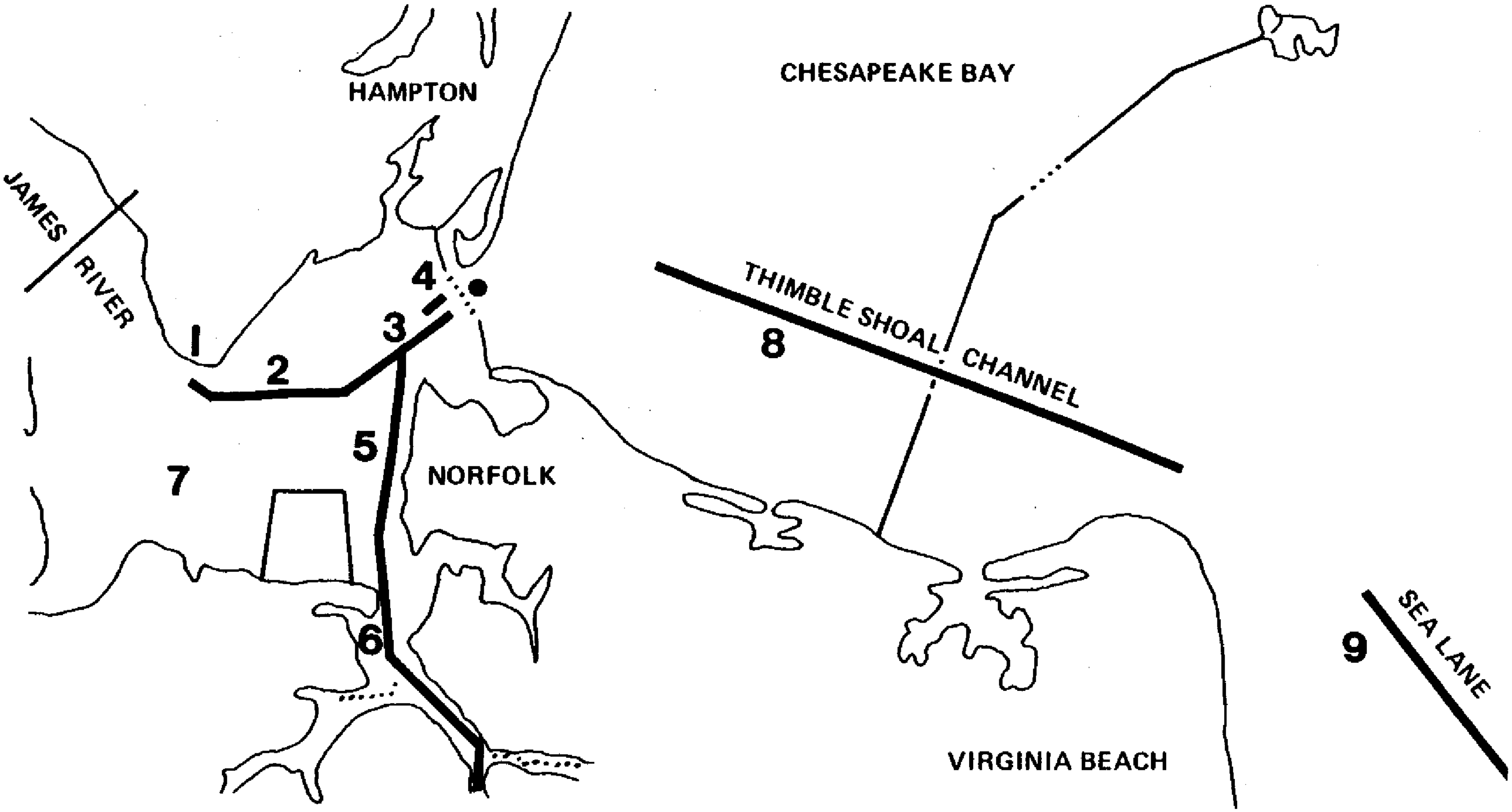
The USACE has noted the importance of examining the entire marine transportation system in the design of U.S. waterways. The recently issued Engineer Regulation 1110-2-1404 states that "Navigation channel design requires careful consideration of human factors in vessel piloting. Human judgment and reactions must be considered in addition to physical design factors. Therefore, optimum channel dimensions for a specific project will require an evaluation of ship maneuverability and pilot or captain response" (1982, pg. 2).

The Norfolk District of the USACE, whose responsibility it is to dredge and maintain federal navigation channels, was the first district of the Corps of Engineers to use simulation in channel design (see Figure 9). In 1981, they became the first district to work in cooperation with the Maritime Administration in the use of CAORF simulation in the Corps design process. The following are examples of the Norfolk District projects. Others are listed in Figure 10.

Hampton Roads Project

The central purpose of this project is to determine channel design requirements for Hampton Roads, Virginia, the largest coal exporting area in the United States. The economics of scale dictate that large, 100,000+ DWT, deep-draft vessels will account for the bulk of future coal exports. The area presently cannot accommodate such vessels fully loaded, so that the port and the USACE are embarking on a large-scale port improvement program to allow deep-draft carriers to be fully loaded at Hampton Roads. These plans include: (1) constructing several new coal loading facilities (two to three in Newport News alone); (2) dredging the major access channels from 45 to 55 feet; and, (3) employing an asymmetrical design by constructing outside lanes on one side of the channel to accommodate fully loaded bulk carriers. Figure 9 provides a chart of the area.

The general objectives of the CAORF component of this program are as follows: (1) to determine navigability of existing channel features by larger ships; (2) to determine maneuvering requirements of large coal-carrying vessels; (3) to determine minimum required widths of the outbound lanes; and (4) to evaluate the effects of the operational restrictions (i.e., wind, current, visibility) that may be necessary.



KEY	
1. Pier 14	6. Craney Island/Lamberts Point
2. Newport News Channel	7. Future Location of Bridge Tunnel
3. Entrance Reach	8. Thimble Shoals Channel
4. Anchorage Z	9. Sea Lanes
5. Norfolk Harbor Reach Channel	

Figure 9. Hampton Roads and Chesapeake Bay with Major Access Channels

Sponsored CAORF Projects

U.S. Army Corps of Engineers

Norfolk Coal Port Development (Norfolk District)
Newport News (Norfolk District)
Thimble Shoals (Norfolk District)
Anchorage Z (Norfolk District)
Newport News Bridge Tunnel (Norfolk District)
Norfolk Sea Lanes (Norfolk District)
Portsmouth, New Hampshire (New England Division)
* Chickasaw Creek (Mobile District)
* Mississippi River Gulf Outlet – Phase II (New Orleans District)
* Mobile Harbor (Mobile District)
* Baltimore Harbor (Baltimore District)

American Association of Port Authorities

Corpus Christi (Port of Corpus Christi)
New Orleans – Mississippi River Gulf Outlet Phase I (New Orleans Dock Board)
Deep Draft Channel Approach (The Port Authority of New York and New Jersey)
* Chickasaw Creek (Port of Chickasaw)
* New Orleans – Mississippi River Gulf Outlet Phase II (New Orleans Dock Board)
* Mobile Harbor (Alabama State Docks)
* Baltimore Harbor (Maryland Port Administration)
* USACE and AAPA Member Cooperative Efforts

Figure 10

The CAORF investigation is a multiphase series of studies examining the entire area. Results from two phases of the Hampton Roads project are highlighted here: the Norfolk Harbor Reach and the Newport News Channel.

Norfolk Harbor Reach

For Norfolk Harbor Reach, the results indicated that the present 500-foot wide channel was inadequate since frequent channel exits occurred. The 750-foot channel was certainly adequate since no channel exits occurred in that channel. The results held regardless of wind, visibility or ship type. The 650-foot wide channel was subsequently tested for adequacy. At this width, only one channel exit occurred and this was in a 12-mile visibility condition, although no exits occurred when visibility was in the 0.5-mile range visibility condition. This one exit appeared to be an anomalous passage because on no other passage did a vessel come within 100 feet of the boundary. A range-marker did not seem to aid pilots in remaining within the channel.

As for vessel controllability, the asymmetrical channel design was not found to hinder controllability. Typical values of swept path were only slightly above the vessel's beam (minimum swept path).

It was concluded, therefore, that the 650-foot wide channel was adequate as a minimum lane width.

Newport News Channel

For Newport News Channel, the results indicated that the entrance at the west end of the Newport News Channel was marginally acceptable. The number of failures (channel exits) was very high in a 2.5-knot current condition regardless of ship type. There were also failures in a 1.5-knot condition as well. A special purpose buoy did not have an impact on navigability, nor did the visibility variable. Based upon the performance of all pilots, a composite envelope and estimate of added dimension requirements for 1.5- and 2.5-knot current conditions were generated with the 225,000 DWT collier.

The eastern portion of the channel was found to be marginal and visibility seemed to be an important determinant. For the southern boundary, the incidence of channel exits was fairly constant across all conditions. At the northern boundary, performance seemed worse in the 0.5-NM visibility condition compared to the 12-NM visibility condition. Again, as in other areas discussed, the specific vessel type did not seem to be an important factor. In addition, more exits occurred in the southern boundary of the channel entrance.

Boundary crossing did not occur in the rest of the channel, thus suggesting that dredging is needed only to flare the east and west ends of the Newport News Channel.

Several other phases for this project were conducted. These phases illustrated the usefulness of the man-in-the-loop simulation approach to examine dredging requirements and cost savings in the Hampton Roads area.

Summary and Conclusions

The marine transportation system is a complex network which, in addition to the operations of ships at sea, encompasses such diverse activities as the operations of shore-side terminal facilities and the design of channels and harbors. The various simulation techniques at CAORF can be extremely useful in designing that portion of the system which requires consideration of the interactions of the man, the vessel and the environment in which they operate.

The results of the cooperative projects highlighted have demonstrated the value of the computer simulation techniques in the design, development and evaluation of port and waterway projects. Simulation permits the interpretation of all relevant variables in the marine transportation system, thus providing important engineering tools with which to study and evaluate alternative system designs.

In recent years, the maritime industry has witnessed dramatic increases in the size and cargo capacity of ships. While advances in shiphandling technology have led to substantial gains in the productivity of shipping, this trend has had some negative ramifications as well. The size and capacity of channels and port facilities have not generally increased in proportion with the size of vessels that must be handled. This created situations in which the relationship between the ship size and channel size left little or not margin for error. While it is vital that economic efficiency of shipping be maximized, this cannot be done at the expense of safety. Thus, it is important to utilize every available tool in designing harbors and waterways to mitigate risks as much as possible.

The paper has also discussed the various engineering simulation tools and how they are used in various stages of the research process. The question most often asked is what is the proper choice of a ship simulator or simulation techniques for a port design problem. An end product that delivers **valid** findings in a cost-effective manner in response to the problem posed is the bottom line. The users must receive meaningful findings and have an acceptable level of confidence in those findings – for example, a high degree of confidence in the recommendation that a 700-foot-wide channel is in fact safe enough! When the investigators err, they must err in the greater-safety direction. This conservative scientific approach is the only acceptable approach, due to the enormous potential adverse impact on safety of inadequate design.

The fidelity of ship simulation system characteristics, and its associated investigative methodology, are the most important factors likely to affect the validity of port design findings. Although some studies have found that under certain circumstances discrete aspects of lesser fidelity simulation can equal the validity of higher fidelity simulation (e.g., black and white vs. color visual scenes in a training scenario), very few studies have shown lower fidelity to be superior. From a conservative scientific viewpoint, therefore, and from the viewpoint of a strong interest in safety, port design should rely on the highest level of simulation fidelity available due to the lack of definitive evidence supporting the validity of lower fidelity simulation.

Research, development and design often deal with unknown effects and interactions. In the absence of other evidence it is prudent to assume that the closer the simulation is to the real world, the less the likelihood of erroneous findings due to uncontrolled variables. This is the premise on which CAORF was built – to minimize unknown compromise in the validity of findings by using the highest level of simulation technology available. As a ship simulation system, CAORF has the overall highest level of fidelity available today. An evaluation of CAORF fidelity is summarized in the following paragraphs.

The only operating mode of a ship simulator of the type discusses herein (i.e., not directly addressing fast-time simulation) is to place the ship operators on the bridge under a variety of simulations. Using rigorous scientific techniques, alternative port design aspects can be validly evaluated in terms of near-real-world ship system performance. Hence, the degree to which the pilots (or deck officers, helmsman) perform, as they would at sea represents the level of confidence that can be placed in the findings. All those characteristics of the simulation that may affect the pilot's performance together comprise the overall level of simulation fidelity. As a simulation system CAORF provides a substantially higher level of overall fidelity than any simulation system in the world.

The ship simulation system can be subdivided into a variety of elements as discussed earlier in this paper. Those for which CAORF has uniquely high capabilities are:

Bridge Fidelity – This is the entire operating environment in which the pilots perform. Overall, CAORF provides a bridge environment uniquely similar to the at-sea situation. By so doing, the pilot's real-time performance closely matches that in the real world, helping to assure a high degree of validity in the conclusions and recommendations.

(1) Visual scene – an exceptionally large (29-foot radius) bridge and detailed visual scene (240 horizontal) that minimizes similarity to the view from an actual vessel, and minimizes potential pilot perception discrepancies that may be associated with smaller screens (such as binocular disparity). For example, complex information in the visual scene abeam may affect pilot/ship performance in the real world by distracting him, or providing additional relevant information. Since definitive evidence is unavailable showing superior validity for a visual scene without a beam view, and since the visual information abeam may affect pilot performance, CAORF's high level of fidelity helps to assure acceptably valid findings. In many instances pilots use only the visual information, as opposed to radar, necessitating high scene fidelity.

(2) Bridge environment – including a large bridge area with actual shipboard equipment and placement (e.g., radars, steering stand, VHF, chart table, etc.). This facilitates the pilots' performing tasks in the same manner they would at-sea. For example, a pilot may walk out on a bridge wing, then have to walk a realistic distance to view the radar, or walk up front to look out the bridge window, and so on. These seemingly minor aspects of the bridge environment (i.e., realistic size requiring walking about the bridge) may have a substantial impact on the pilot/ship performance. Research studies on bridge design have found that a small cockpit-type bridge where the pilot stays in the small area results in changed shiphandling performance. Hence, when dealing with large vessels having a large bridge, a more realistic large simulator bridge would be expected to yield more realistic pilot/ship performance. Many other aspects of the bridge environment (e.g., equipment) should be viewed similarly.

(3) Scenario fidelity – the scenarios used in port design must be realistically representative of the range of situations likely to be encountered at sea. This is important for loading the pilot, and for providing him with realistic situations dissimilar to those he will likely encounter at sea. For example, pilots often readily identify ferryboats crossing ahead, or tug activity around an anchored vessel that has yet to notify his intention of departing. The scenarios should also represent the most difficult situations likely to be encountered, such as passing or overtaking other large vessels. The highly capable CAORF simulation system enables the use of highly complex and realistic scenarios, thus achieving realistic pilot tasks, workload and performance.

Other aspects of bridge fidelity unique to CAORF could be listed. The purpose of the complex combination of characteristics that yield bridge fidelity on the simulator is to structure a highly realistic environment that addresses the variety of subtleties inherent in pilotage. This helps to assure that the pilot will perform most similar to at-sea – the real-time sequence of the precise tasks he would perform at sea, with the concomitant difficulty, rate of performance, and so on, that would be most like his at-sea performance after the port design modifications are made. Although all ship simulations have aspects of compromise, without definitive evidence to the contrary, the more similar the simulation is to the real world the greater the confidence in port design conclusions and recommendations.

Scientific Methodology – CAORF has pioneered the used of rigorous objective scientific methodology in ship simulation research and development. CAORF uses a wide range of analytical tools to design experiments, collect data and analyze findings. These greatly contribute to the confidence and generalizability of conclusions and recommendations. Relevant aspects of the CAORF approach include:

(1) Sophisticated experimental design and analysis techniques (parametric and non-parametric statistics) are carefully tailored to the needs and objectives of each port design project. This assures a high yield of valid findings on a cost-effective basis.

(2) Scenarios, operating procedures and data collection techniques are carefully configured to be as realistic as possible, with minimum change influence on the pilot's normally available information and actions. For example, the data collectors are usually not present on the bridge during a data collection scenario, so as not to indirectly affect the pilot's performance, the pilot's activities are remotely monitored in a non-invasive manner.

(3) Only properly qualified pilots of the port understudy are used as subjects in port design studies, and then always in sufficient number to yield statistically significant findings. This is critical to the validity of the findings, taking the simulation system from the realm of a game to that of the actual real-world piloting situation.

(4) Analytical techniques center around three major parallel methods: (1) sophisticated objective performance measures generated from data automatically collected by the simulator (e.g., swept path of the ship moving through a channel bend); (2) direct human performance data collected by qualified observers (e.g., time-line rudder usage by the pilot); and (3) structured subjective evaluation of vessel track plots by appropriately qualified individuals (e.g., pilots). The results of each of these methods are correlated to better detect meaningful performance differences, and to yield higher confidence in conclusions and recommendations.

CAORF Staff – the CAORF staff design is configured to provide an unusually broad range of expertise in simulation, the technical discipline necessary to conduct ship simulation work, and the applied areas in which ship simulation is used. The staff is structured, however, to permit flexible assignment of individuals to CAORF projects on a demand basis.

This approach provides the range of expertise associated with a large organization to expertly tackle a wide variety of problems, while at the same time achieving the economics of operation associated with a relatively small organization. The approach is centered around the structure of a core staff and resource pool.

Validation – A particular strength at CAORF is the extent to which it has been validated – probably more so than any ship simulation in the world. Prior to initiating operation as a research facility in 1974, several studies were accomplished to evaluate the validity of various aspects of this simulation system. For example, mathematical hydrodynamic ship models were validated via towing tank tests, comparison with the models of others simulators, and on-the-bridge testing by pilots. Furthermore, the validity of the human performance component of the system was evaluated by correlating ship and deck officer performance at sea with their corresponding performance in duplicated 4-hour watches on the simulator. Additional validation studies have been conducted over the past 10 years, including comparison of ship model hydrodynamics with actual at-sea ship performance (e.g., ESSO OSAKA, Panama Canal vessels). These validation efforts have resulted in many changes to CAORF to improve the validity of its research products. Although CAORF has received substantial validation attention, considerable work remains to be done to definitively relate aspects of the ship simulation system and fidelity to the validity of port design conclusions and recommendations. The lack of sufficient validation information strongly urges the use of the highest-fidelity simulation available, to err on the conservative side and thus enhance confidence in the conclusions and recommendations. When lower levels of simulation fidelity are proven acceptably valid for certain areas of investigation they can then be used for those specific areas to maximize cost-effectiveness.

In conclusion, CAORF represents the high end of ship simulation fidelity. This high end includes not only the simulation, but also the staff and scientific methodology (i.e., use of pilots, design scenarios, etc.). In essence, the total CAORF environment is as close to the at-sea environment yet achieved in ship simulation, and likely would result in pilot/ship system performance equally close to the expected at-sea situation. The simulation data collection and analysis capabilities and the broad staff expertise enable a depth of analysis to achieve a comprehensive picture of design alternatives, and provide a high level of confidence in port design conclusions and recommendations.

Desk-top ship simulation is at the other extreme of fidelity, with medium-fidelity ship simulation falling in between. These lesser fidelity levels have the attractiveness of lower-cost research, although at a higher risk if their validity in the particular area of research has not been definitively established. For this reason, extreme caution must be used in conducting port design research with medium- or low-fidelity ship simulation. Math models describing ship behavior may be of sufficient fidelity, but the shortcomings in environment replication and accommodation of the variety of subtleties inherent in pilotage can have a significant impact on simulation validity. In general, the lower the fidelity of ship simulation, the more important its validation.

CAORF has cooperated with many segments of the marine community to conduct waterway design analysis. Through this work, CAORF has developed an evaluation approach to design assessment which is based in the collection of objective data utilizing a high-validity research tool (CAORF simulator) within the framework of rigorous method. The integration of this data with subjective evaluations by experts (e.g., pilots, ship masters, operations personnel) associated with the specific areas under investigation provides a thorough evaluation of a proposed design. Thus, simulation has proven to be a most valuable tool in port and waterway engineering.

It has been recognized that simulation supplies data which is essential for making credible, real-world decisions. It is generally the case that the maritime simulation facility (CAORF) acts as a project facilitator by providing data which is quantitative and unambiguous. Through cooperative efforts, the simulations performed and analyzed yield results which guide planners and decision makers in assessments of the trade-offs inherent in the continuing necessity to build, dredge and implement improvements to ports in a cost-effective manner.

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Biodata

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Joseph J. Puglisi is presently the managing director of the Computer-Aided Operations Research Facility (CAORF) in the U.S. Maritime Administration's Office of Advanced Ship Operations. He is in charge of projects in the areas of advanced ship operations, port and harbor design, computer simulation, electronic navigation and communications. He joined the Maritime Administration in 1969 at the U.S. Merchant Marine Academy, Kings Point, New York, on the Savannah Nuclear Power Plant Simulator Program. When the National Maritime Research Center (NMRC) was formed in 1971, Mr. Puglisi was assigned as project director on the Maritime Satellite NAV/COM System Experimental Program. He has been involved with CAORF since its inception in 1971. He was previously employed as an engineer with the Naval Applied Science Laboratory in Brooklyn and the Naval Ship Research and Development Laboratory in Annapolis.

Mr. Puglisi received a B.S. degree in electrical engineering from City College of New York, an M.S. degree from New York University and has completed postgraduate courses in a Ph.D. program in computer science and operations research at Polytechnic Institute of New York. Mr. Puglisi is a member of the Society of Naval Architects and Marine Engineers (SNAME), and the Institute of Navigation. He is a member of the Society of Computer Simulation, a past member of their board of directors and an associate editor for marine applications. He is also the present chairman of the International Marine Simulator Forum (IMSF), an organization which provides operators and users of complex ship simulators with assistance in the effective use of their respective national resources to analyze ship/port transportation systems.

The Use of Simulation Techniques for the Development and Validation of a Proposed Widening Solution for the Gaillard Cut Section of the Panama Canal

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Introduction

The ultimate objective of a waterway design/improvement project is to allow the safe and efficient movement of vessels from one point to another (for example, between the open sea and some commercial terminal). While channel designs can be evaluated mathematically with respect to the inherent capabilities of vessels, the final analysis must consider the ability of the human navigator to perform his task within the proposed design, under a variety of operational and environmental conditions. In this respect, simulation, particularly man-in-the-loop real-time simulation, has become an indispensable tool in the cost-effective design of channels, ports and harbors.

Simulation provides an environment in which new channel designs can be tested safely, under the most severe environmental conditions or emergency situations, before these new designs are committed to actual practice. In addition, extremely precise measurements of ship performance can be made relatively inexpensively and unobstrusively when compared with the effort needed to collect such data in the real world. This data can then be used to evaluate the adequacy of the proposed waterway layout quite specifically, and to address any shortcomings through refinements to the design. However, the utility of data collected through simulation is completely dependent upon the validity of the research methodology, which in turn is affected by the research tools used in a simulation project. The execution of a simulator research project is a complex process with many interrelated phases involving development, validation and execution. The purpose of this paper is to describe and illustrate this process, as applied to the Gaillard Cut Widening Study, undertaken for the Panama Canal Commission (PCC) between December 1983 and September 1986, and perhaps the most comprehensive simulation undertaken to date at the Computer-Aided Operations Research Facility. This project was especially involved in that it was one of the few projects using simulation for the design as well as the evaluation of alternative waterway layouts. Because the study is unique in this respect, this paper will place particular emphasis on that aspect of the research.

Overview of Simulation

Real-time, man-in-the-loop simulation allows the channel designer to assess the entire man/machine/environment system. In order to do this successfully, each aspect of the system must be carefully selected, developed and validated in its own right, while keeping in mind the interaction between the elements, and the overall objectives of the study. In general, the following elements are involved in the simulation study:

- ship(s)
- navigators/shiphandlers
- waterway(s)
- environmental influences

To ensure the applicability of the simulation results to the real world, these elements must be carefully selected according to the program objectives, modeled for simulation, then validated against real-world performance. In addition, it must be considered that a proposed waterway must be safe under a range of situations in the real world. Therefore, the following factors must also be taken into account:

- operational conditions
- environmental conditions

This is accomplished by studying the range of credible ship states (e.g., speed, loading condition, navigation equipment) and environmental conditions (e.g., wind, current, visibility) under which shiphandling is expected to take place in the real world. These conditions, or a representative subset, must then be included in the simulation.

Finally, in order to perform the evaluation of a waterway in a meaningful fashion, measures of performance must be developed which address the degree to which the objectives of the study have been met by a given layout. The evaluations must also be carried out using rigorous experimental methodology so that the level of performance can be associated directly with the layout (or other factor of interest) and not with some other irrelevant variable. Each of these issues was addressed in the Gaillard Cut Widening Study, as described in the following sections.

Objectives of the Gaillard Cut Widening Study

In December 1982, the Panama Canal Commission initiated a project at CAORF intended to develop recommendations for cost-effective improvements to the Gaillard Cut section of the Panama Canal. At present, the Gaillard Cut is approximately 500 feet in width throughout. Given this width, the PCC has determined that for safety reasons, when Panamax vessel is in transit through the Cut, all ships traveling in the opposite direction must be held outside the Cut. This "clear-cut" restriction has become problematic from an economic standpoint as the number of Panamax ships traveling through the Canal has increased dramatically in recent years and is projected to continue increasing into the foreseeable future. While it is intuitive that widening the channel in general permits safer meeting encounters between larger vessels, it must be borne in mind that widening past a certain point can result in greatly increased excavation costs without appreciable safety increment. The purpose of the Gaillard Cut Widening Study was to determine the dimensions and configuration of improvements to the Gaillard Cut which would allow two-way Panamax traffic without involving unnecessary effort or expense.

While real-time simulation with a man-in-the-loop provides a powerful tool for use in the evaluation of waterway designs and modifications, channel design can be a complex process in which many levels of the relevant parameters can be combined to form large numbers of possible layout alternatives. In addition, large, carefully composed samples of pilots must be tested in order to ensure that the results are generalizable to the population at large. If every possible layout alternative were tested under these conditions, the time and expense involved could easily become prohibitive. Therefore, some means are needed to narrow the range of possible channel configurations, and eliminating from consideration those that are clearly unacceptable.

The methodology used for narrowing the range of possible alternatives was a computer-controlled mathematical modeling of ship behavior by means of which the results of a shiphandling run could be calculated more quickly than in real time. While the vessel's position change from time"x" to time "x" + 10 seconds would take 10 seconds to process in real time, by definition, the same position update would be calculated in fewer than 10 seconds in compressed-time. The exact processing time varied as a function of the complexity of the scenario which the vessel was required to complete. For example, a meeting in a straight reach which took 12 minutes to complete in real time required approximately 5 minutes to execute using the compressed-time program. In conjunction with the ship behavior model, a model of the human navigator had to be developed which could simulate the decision actions of the human pilot. Finally, even though this model could perform runs more quickly than in real time, so many layouts were possible that every one could not be tested individually. Therefore, a system was needed by which groups of layout alternatives could be eliminated based on inferences drawn from a small sample of direct tests. This overall approach, referred to as the "compressed-time analysis," provided the framework which supplied the specifications for all aspects of the project. The compressed-time approach is described in detail in the following section.

Compressed-Time Analysis: General Approach

As stated previously, simulation projects require the specification and/or modeling of ships, shiphandlers, waterway designs, and the operational and environmental conditions expected in the real world. Furthermore, specific measures are needed for assessing the adequacy of performance in a proposed waterway layout. In the compressed-time analysis, a sample of the possible layouts for different areas of the Gaillard Cut was tested to eliminate those layouts that were clearly unacceptable. The first step in the analysis was therefore to develop specifications for the layouts to be investigated, and to construct simulated models of these layouts. In addition, a criterion for assessing the acceptability of the layout alternatives was needed, as well as measures for comparing actual performance to the criterion. Finally, a procedure for executing the tests was developed which made the most efficient use of computing resources while providing the valid inferences regarding every possible layout alternative.

Specification of Conditions for Investigation

Since the overall idea of the compressed-time analysis was to identify a small number of conditions for possible testing on the full fidelity simulator, the first step in the analysis was to specify the domain from which the ideal alternative might be selected. This involved describing the parameters which define a curve layout, and the range of values to be tested for each parameter. Also, the state of the vessels in the curve was defined by a set of parameters, each with a range of values. The combination of all possible vessel states in each of the alternative curve layouts constituted the sampling domain. The parameters and associated values are outline below.

Layout Alternatives

The Gaillard Cut section of the Panama Canal is approximately nine miles in length, and incorporates eight curves of varying sharpness connected by straight reaches. To simplify the analysis, each curve was investigated individually, along with one representative straight segment. In the straight reach, only one layout parameter was manipulated. This parameter was the width of the channel and was conceptualized as the distance between two parallel straight banks. Beginning with the present 500 feet, the width was increased in 50-foot increments to a maximum of 750 feet.

Three parameters were identified for defining alternative curve layouts. An alternative layout is described by combining one level of each of the parameters; the total number of alternatives for any one curve is determined by multiplying through by the number of levels for each parameter. The layout parameters are as follows:

- Radius of Curvature (see Figure 1a) – In a turn, the centerlines of the two adjacent straight reaches are connected by an arc which is tangent to both. This arc is the trackline which the vessel is assumed to follow, and its radius is the radius of curvature. This determines the shape of the inner bank because the bank is dredged along an arc concentric with the arc connecting the centerlines. Six levels of radius of curvature were tested for the improved channel, beginning with the present radius for the turn and increasing in 500-foot increments.
- Width (see Figure 1b) – In a turn, the width is defined as the distance between the banks along a line, perpendicular to the channel centerline, through the point at which the arc of the curve is tangent to the straight reach. The width began with that selected for the improved straight section, and increased in steps of 50 feet with a maximum of 6 increments.
- Transition Zone Slopes (see Figure 1c) – In those cases where the width of the curve exceeded that of the straight section, a transition zone was used to connect the two. The slope of the transition zone was defined as the ratio between two distances: (1) the distance from the curved bank end to the straight bank along the line perpendicular to the point of tangency of the curve arc, to (2) the distance along the straight bank at which the transition zone intersects it. Three slopes were used, namely 5 percent, 10 percent and 20 percent.

Operational Conditions

In planning a waterway design, it must be recognized that real-world vessel transits will not occur under the same conditions in every instance. Therefore, it is important to demonstrate that a proposed design will permit safe navigation in a realistic variety of situations. In this study, these situations were defined in terms of a set of dimensions along which one meeting encounter might differ from another in the real world. A range of values for each dimension was then determined via discussions between CAORF staff, PCC technical staff and PCC pilots. A representative sample of real-world meeting situations was then defined as every possible combination of all levels of the important parameters. The parameters used, and the values associated with each are described in the following sections.

Speed. The speed at which the two ships are traveling during a meeting encounter is an extremely significant factor influencing safety, particularly because of the associated effects on bank and passing ship forces in a restricted channel.

In this study, several speeds were chosen to represent different ways in which a pilot might approach a meeting encounter. One possible approach would be for a pilot to maintain his normal transit speed throughout the encounter. To represent this case, a condition in which both ships were moving at six knots was included.

At the opposite extreme, a pilot might substantially slow the vessel well prior to the meeting, and proceed through the encounter slowly to minimize hydrodynamic disturbances. A condition in which both ships traveled at four knots was therefore included as well.

Finally, it is expected that in many cases the pilot will proceed at normal transit speed until the other ship is in sight. At that time the pilot will allow ownship to slow somewhat until the vessels meet and then raise

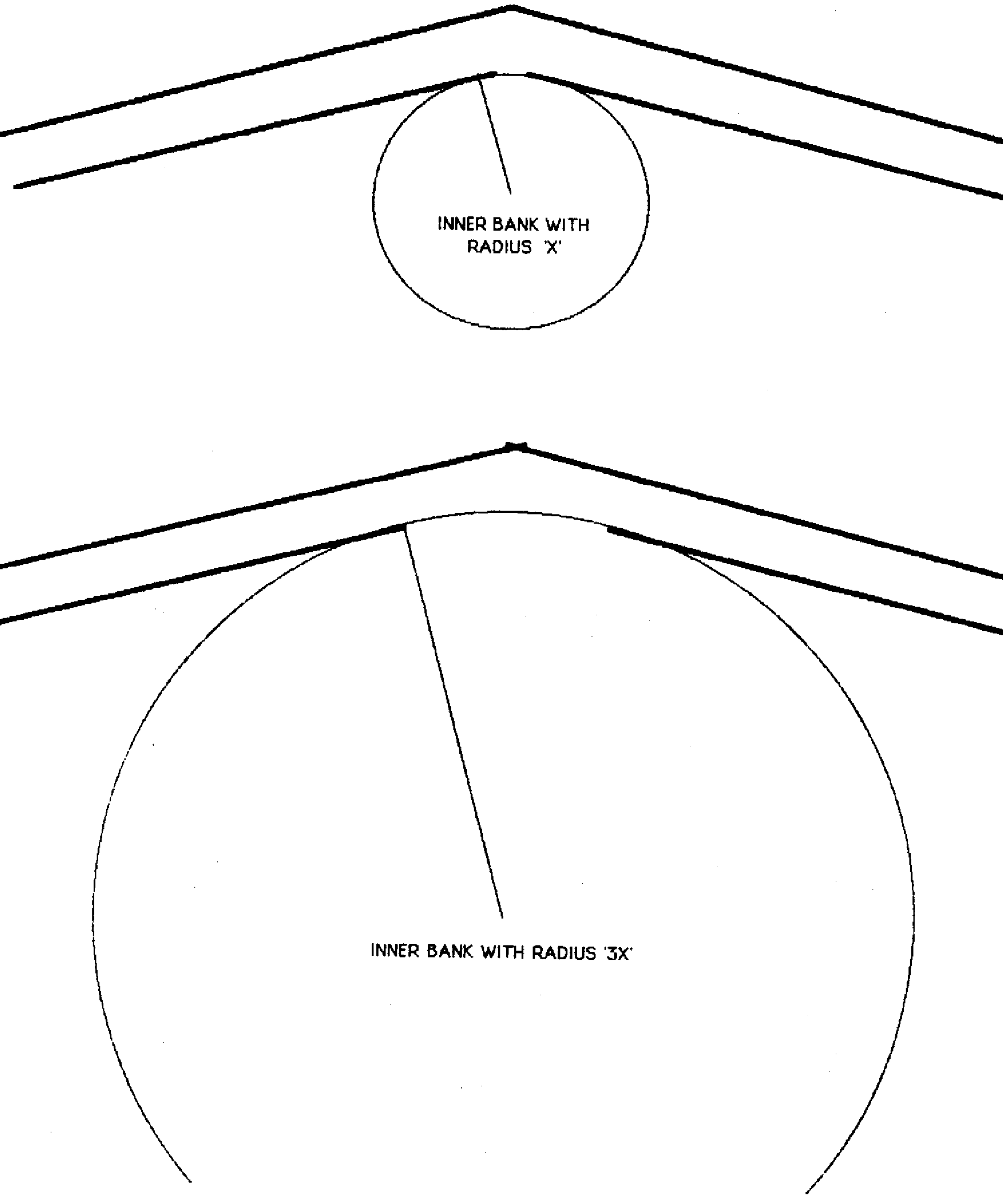

Figure 1a. Illustration of Inner Banks with Varying Radius of Curvature

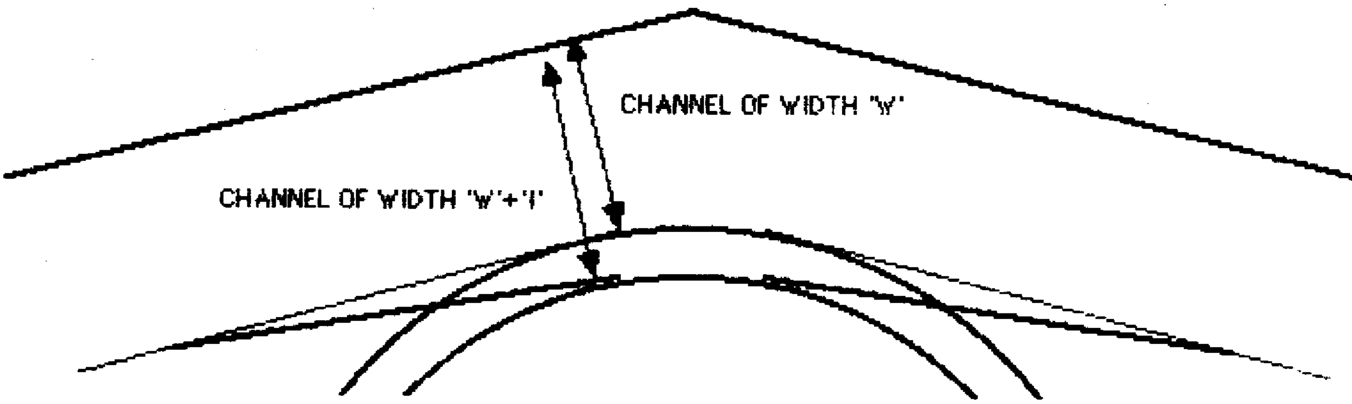

Figure 1b. Illustration of Inner Banks with Varying Width

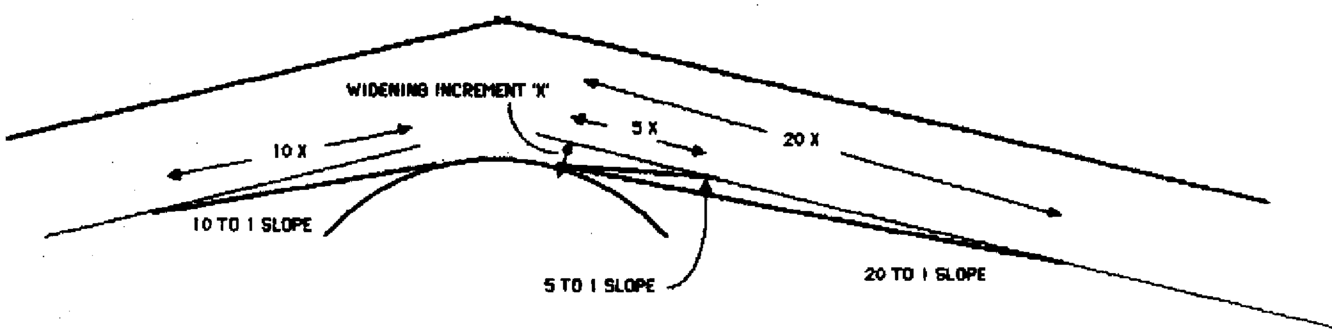

Figure 1c. Illustration of Widened Turn with Varying Transition Zone Slopes

engine RPMs to increase control force. This approach was approximated by a condition in which the ships were initiated at six knots but with engine RPMs sufficient to support five knots.

Cross-Track Starting Position. In the course of normal operations, there will be some variation in the cross-track position achieved by the pilots as transits are made. Therefore, the cross-track position of the vessel must be expected to vary at the moment a meeting encounter begins. This variation in turn can be expected to affect performance in the meeting; therefore, it was considered important to account for differences in initial position in designing the waterway. Four cross-track positions were used.

• Centerline – To represent a "normal" situation, the ships began the scenario on the channel centerline (see Figure 2a). At a certain distance into the run (specified by the experimenter), the ships moved to the sailing line to complete the meeting maneuver.

• Sailing Line – In some cases it can be expected that pilots will go into a meeting soon after having completed a turn or a previous meeting encounter. To simulate such an event, the vessels began the encounter on their respective sailing lines. The sailing line is the term used to describe the ideal path which the vessels will follow in performing a meeting encounter (see Figure 2b).

• Between Sailing Line and Bank – In order to adequately assess the safety of a waterway, some consideration must be given to the fact that encounters may occur when ships are not placed precisely as the pilot intends. To account for this, the proposed layouts were tested in situations which can be considered somewhat out of the range of normal operating conditions. In the first of these conditions, the vessels began halfway between the sailing line and the bank (see Figure 2c).

• Between Centerline and Opposite Sailing Line – This condition was included as an extraordinary situation similar to the previous condition. In this case, the vessels began the encounter on the wrong side of the channel; specifically, half the distance from the centerline to the sailing line (see Figure 2d).

Anticipation Distance. The final condition used in the study was intended to deal with differences in piloting style. This stylistic difference was operationalized as the distance prior to the meeting at which the pilot moves the vessel from the centerline to the sailing line. According to the PCC pilots consulted in this project, some pilots move the vessel to the sailing line as soon as they become aware that a meeting is going to occur, while others remain close to the centerline and let the bow wave of the traffic ship move them to the side. A sample of eight PCC pilots each performing several runs were observed on the CAORF simulator and conclusions were drawn regarding the anticipation distances used. One distinct grouping was observed at 12 shiplengths before bow-to-bow, and another at eight shiplengths. These two distances were used as the levels for this parameter. Figure 3 illustrates the anticipation distance factor.

Summary of Operational Conditions. An operational condition for a given run was defined as the combination of one level of each of the three parameters described above. The levels of each of the parameters are summarized as follows:

• Speed – 4 knots vs. 5 knots vs. 6 knots

• Initial cross-track position – centerline, sailing line, between sailing line and bank, between centerline and opposite sailing line

• Anticipation distance – 12 shiplengths before bow-to-bow vs. eight shiplengths before bow-to-bow.

The total number of operational conditions to be investigated comprised all possible combinations of the levels of the above parameters. Multiplying three speeds by four cross-track positions by two anticipation distances yielded a total of 24 conditions. Table 1 summarizes these conditions.

Development of Decision Strategy

In order to choose the optimal channel design, the performance of meeting vessels under each of the 24 operational conditions had to be evaluated in every plausible layout alternative. The latitude of operational conditions under which acceptable performance could be expected could then be determined for each layout, and this latitude weighted against the associated excavation cost. The trade-off between cost and latitude of performance was used by the PCC in making the final design selections.

In defining crucial layout parameters and the range of values to be examined, a domain of 108 layout alternatives for each curve was specified (6 radii by 6 widths by 3 transition zones). Testing the 24 operational condition in every layout would have required 2,592 runs to be performed for each of the eight curves in the Cut. Even with the speed advantage of the compressed-time analysis, however, 2,592 runs for each curve could not be carried out within the time and financial copse of the study. Therefore, a decision strategy was devised to allow interferences regarding the latitude of acceptable performance within each layout alternative, without the need to test every possible scenario.

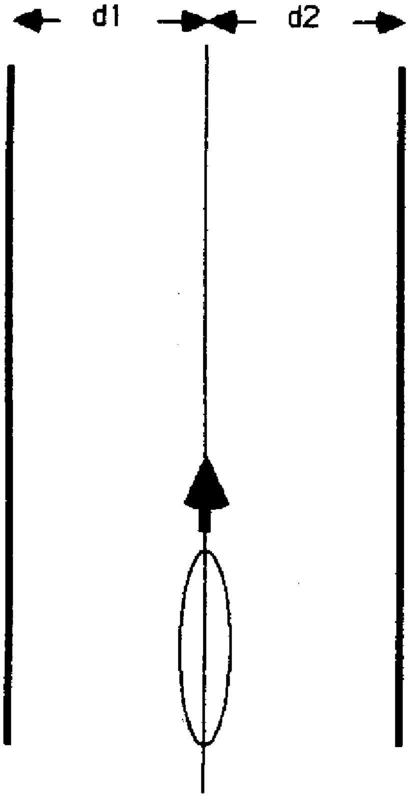

Figure 2a -beginning on centerline (d1=d2)

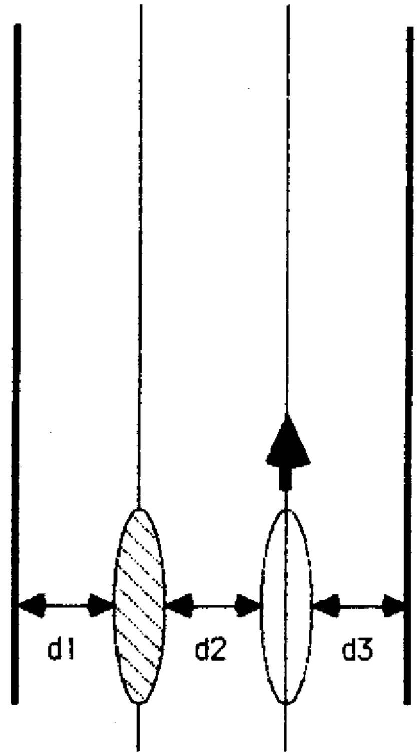

Figure 2b - beginning on sailing line (d1=d2=d3)

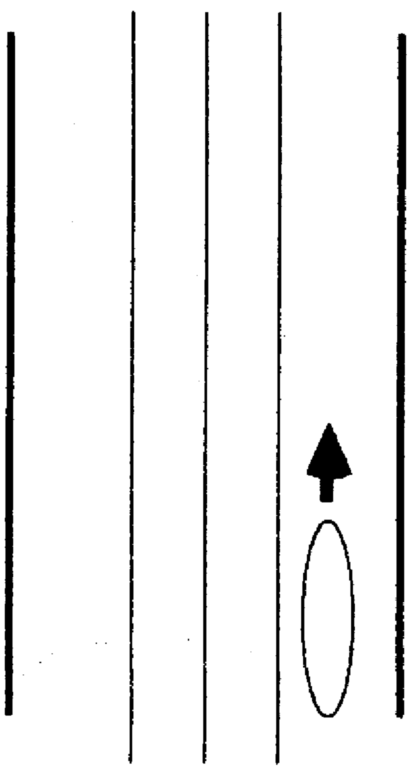

Figure 2c - beginning between sailing line and bank

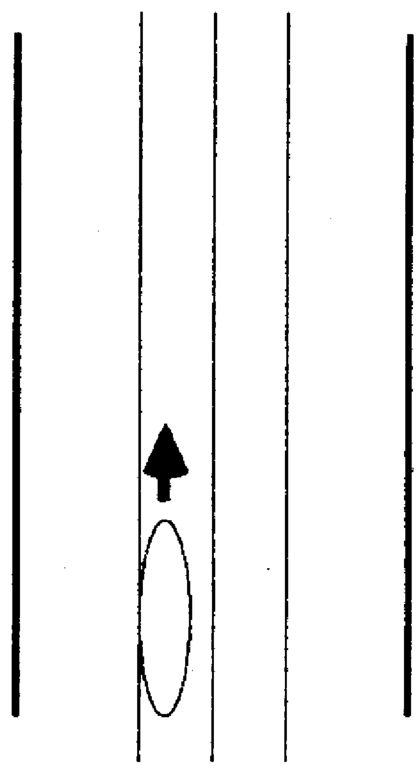

Figure 2d - beginning between centerline and opposite sailing lin

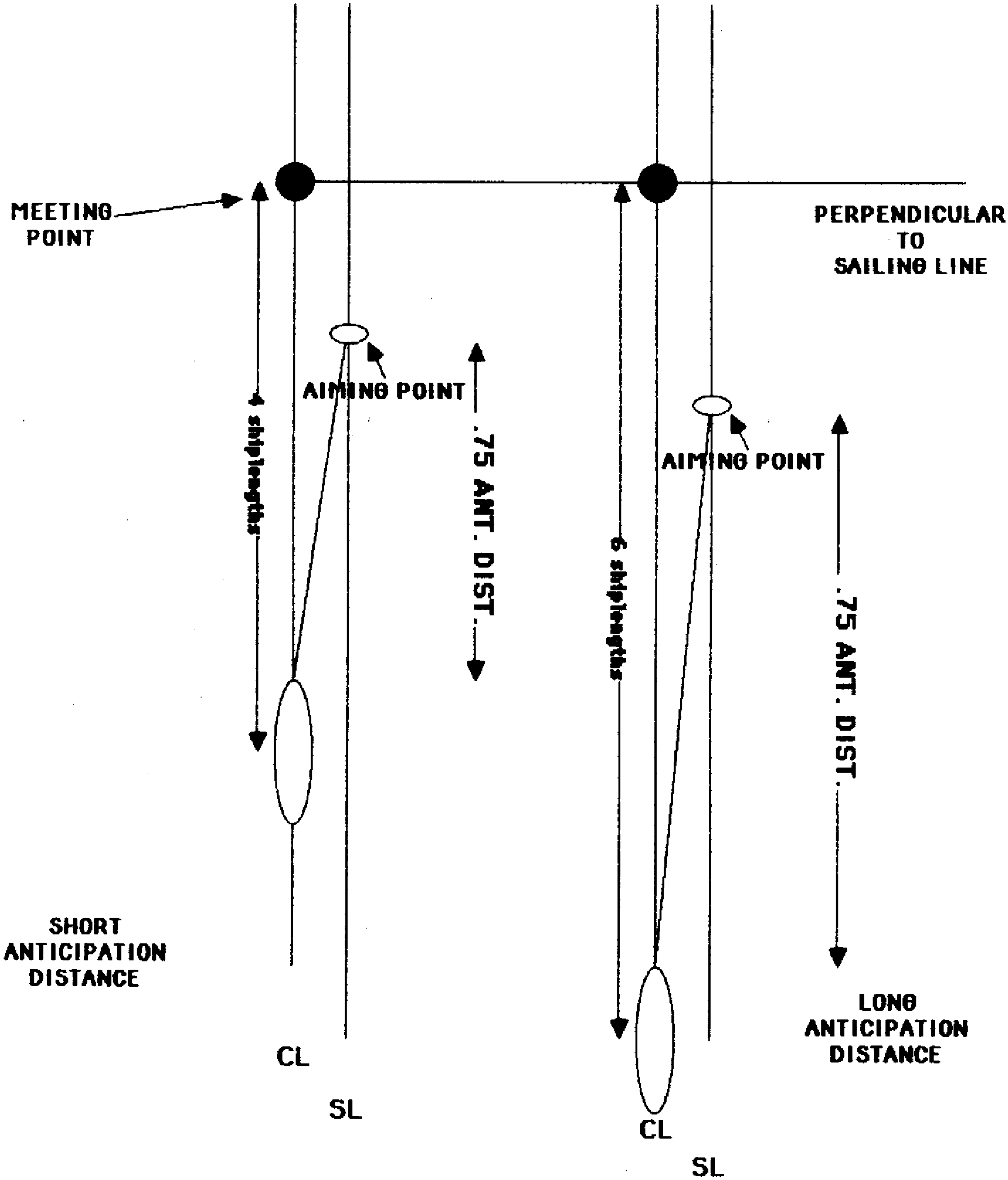

Figure 3. Illustrations of Vessel moving to Sailing Line with Short vs. Long Anticipation Distance

Dimensionalization of Problem Space

The strategy was based on the idea that decision problems can be broken into a set of major factors; in this case, the channel layouts and the operational conditions of the vessels. Dimensions were then determined which a) were relevant to the study problem, and b) could be used to discriminate individual cases of the identified factors. By organizing the instances of the factors along these dimensions, the results of any given run could be used to predict the results of runs performed under other conditions. This section details the rationale underlying this strategy.

Excavation Cost of Layout Alternatives. The dimension used to organize the layout alternative factor was the cost of implementation (operationalized in terms of the volume of excavation required). This dimension was utilized on the reasoning that once a layout was identified which allowed acceptable performance, other layouts involving more excavation would no longer be of interest. Thus, these alternatives did not have to be tested directly, and effort was substantially reduced.

Effectiveness of Operational Conditions. The operational conditions in this study were arranged according to their "effectiveness," or the quality of the performance they could be expected to yield. In this way, if acceptable performance was obtained with a given operational condition, it was assumed that all more-effective operational conditions would also pass in the same layout and therefore did not have to be tested.

Implementation of Search Strategy

The dimensionalization effort described in the previous paragraphs provided the basis for an efficient search for the optimum layout solution. Since the layout selection involved balancing implementation cost against operational flexibility, the compressed-time analysis was used to identify the operational conditions for which acceptable performance could be obtained within each of the layouts in the possible set. This objective determined the conceptualization and development of the search strategy.

The search began with the least costly layout and the most effective set of operational conditions. If acceptable performance was not obtained, then layouts requiring increasingly more excavation were tested until criterion performance levels were met. The layout thus identified was important as an anchor point in that it established the least amount of excavation that would have to be performed before acceptable performance could be achieved under any circumstances.

Following the identification of the least necessary amount of excavation, the search strategy evaluated the range of operational conditions that would pass within that layout. Progressively less effective sets of operational conditions were tested within the chosen layout until a failing condition was reached. In this way, the latitude of passing operational conditions in the first layout alternative was identified.

The progress of the search strategy through the remainder of the problem space consisted of iterations of the procedure outlined in the preceding two paragraphs. Having arrived at the first operational condition which would not pass in the layout under examination, the search program tested increasingly more costly layouts until acceptable performance was obtained for that operational condition (the more effective operational conditions did not have to be tested in this new layout, because of the assumption that if a given operational condition passed in a layout then all more effective operational conditions would also pass). Progressively less effective sets of conditions were then tested in the new layout until passing performance was no longer obtained. In this way, the performance latitude for that layout was established, at which time the search resumed testing more costly layouts in the manner described above. This procedure continued until a layout which would allow acceptable performance under the least effective set of operational conditions was identified.

The output of the search provided a list of alternative layouts, each associated with a group of acceptable operational conditions. It is important to note that the group of conditions associated with a particular layout are in addition to those passing the next less costly layout. Therefore, the increase in excavation cost required to increase operational latitude could be readily determined. A brief example illustrating the utilization of the decision strategy is provided in Appendix A of this paper.

Criteria

The practicability of the search strategy described above depends to a large degree on being able to evaluate the results of a shiphandling run as "acceptable" or "unacceptable." In other words, a given operational condition must be characterized as either "passing" or "failing" in a particular layout in order for the search strategy to select either the next operational condition or the next layout alternative, respectively.

This determination required a) the establishment of a specific criterion level of performance, and b) the development of measures to be used in comparing observed performance with the criterion. This section outlines the fulfillment of these two requirements.

Selection of Safety Criterion

In conceptualizing a baseline safety level, it was decided to utilize some aspect of shiphandling, currently exhibited by the PCC pilots, which is generally considered to constitute acceptable performance. Given that Panamax ships will be the largest vessels passing in the improved Gaillard Cut, it was decided that an appropriate criterion performance would consist of the manner in which pilots carry out meetings in the present Cut aboard the largest ships which are now allowed to pass others of the same size. A generic bulk carrier of 33,000 DWT, 608 feet in length and 85 feet in beam, was selected for this purpose. This ship will hereafter be referred to as the "validation ship."

This type of ship was selected for study for the following reasons: 1) such ships make frequent transits through the Canal and thus provide an example of performance which is relevant to the real-world situation; 2) extensive real-world performance data is available; and 3) Dr. Haruzo Eda, hydrodynamic consultant from Stevens Institute, has had previous modeling experience with this vessel class. This ship was modeled based on coefficients taken from the Series-60 bulk carrier, supplied by Dr. Eda. Extensive model validation was then carried out using real-world ship trajectory data collected in the Gaillard Cut, and the subjective opinions of pilots on the simulator. Three basic techniques were used to validate the performance of the validation ship:

- trajectory matching
- rudder order evaluation
- unconstrained maneuvering

Trajectory Matching. The trajectory matching approach involved first collecting data from trackkeeping and meeting encounters done aboard two 85-foot-beam vessels in the Gaillard Cut. Precise measurements of trajectory, heading, rudder angle and ship separation distance were made periodically by helicopter, shoreside triangulation and shipboard observation. Pilots then performed identical meeting encounters using the simulator model, keeping as close to the trajectory of the real-world vessel as possible, and the rudder activity was compared. The simulated ship required 13 degrees of rudder to stay on the sailing line at 6 knots versus 11 degrees for the real-world ship. The maximum rudder used during the meeting was 20 degrees right for both ships. It was concluded that simulator and real-world results were comparable considering differences in the specific ships and piloting systems.

Rudder Simulation Methodology. For this approach, the rudder commands given by the PCC pilots during real-world trackkeeping and meeting exercises were recorded on microcassette, as precise measurements of ship trajectory were made via shoreside triangulation. The validation ship model was then placed int he same initial location in the simulated Gaillard Cut, and proceeded through the Cut while the recorded rudder commands were executed. Comparisons were then made between trajectories of the simulated and real-world vessels.

The results of the trackkeeping scenario resulted in a maximum heading difference of less than 1 degree, and a maximum position difference of 12 feet over the course of a .7 Nautical Mile (NM) transit. For the meeting scenario, the maximum heading difference was 2 degrees, and the maximum position difference was 20 feet for a .5 NM transit. The performance of the simulated vessel was in close agreement with the real world vessel, considering measurement accuracy, differences between ships, and the sensitivity of the methodology to initial conditions and the timing of the rudder commands.

Unconstrained Maneuvering. Two PCC pilots conned the validation vessel through the simulated Gaillard Cut after all fine tuning was completed. These transits included meeting encounters similar to those experienced by the real world vessel in the scenarios described in previous sections.

The simulated transits resulted in overall differences from the real-world vessel of 2 degrees in heading, 1 degree in left rudder, Room Mean Squared (RMS), and 4 degrees of right rudder RMS. The difference in the ratio of left to right rudder between simulator ship and real-world ship was 0.3. These differences are not considered significant in view of differences between specific ship types and piloting styles. Furthermore, most of the observed differences were attributable to Pilot 2, who showed greater variability in speed, and also stayed closer to the bank, thus requiring more right rudder.

These techniques were performed independently, and used entirely different conceptualizations in validating the ship model against real-world performance. The use of multiple method convergence yields a much more powerful assurance of the validity of the model than would be obtained through any single methodology. This level of effort, while quite involved and time-consuming, was considered necessary since the utility of any simulation study rests squarely on the validity of the modeling.

The second ship modeled for this study served as the Panamax vessel for which the improved layouts were being designed. Since the objective of the study was to allow meetings between two Panamax ships, it was of crucial importance that the model used in the simulation be accurate in terms of its control responses and the degree to which it was affected by the bank and passing ship interactions experienced in an enclosed waterway.

The ship selected for modeling, the San Clemente Class ULTRAMAR, was a bulk carrier of 894 feet in length and 106 feet in beam. This vessel was one for which the substantial at-sea maneuvering data was already available and which could be used to validate the CAORF model. In addition, the course recovery characteristics of this vessel were felt to represent an especially poor-handling situation, such that if satisfactory performance were obtained for this ship then the majority of other ships could also be expected to exhibit satisfactory performance under similar circumstances.

An extensive program of physical model-testing was then conducted at the SSPA Maritime Research Center in Goteborg, Sweden, to develop ship-bank and ship-ship interaction coefficients. These coefficients were calculated at several channel widths, separation distances, and depth-to-draft ratios to increase the generalizability of the model to various channel configurations.

The coefficients generated at SSPA were converted for use in the CAORF model by Dr. Haruzo Eda of Stevens Institute in conjunction with CAORF staff. The performance of the model so constructed was compared with at-sea handling data from the actual vessel, and the model constructed at SSPA to validate the accuracy of the model. Also crucial to the validation process were the comments of experienced PCC pilots who took the model through a series of trackkeeping maneuvers in the simulated Gaillard Cut. Several adjustments were made to the bank interaction effects to bring the ship into closer agreement with the pilots' subjective assessments. In addition, observations of the rudder activity needed to counteract bank effects in the Gaillard Cut were performed by the PCC, and were utilized alongside the other data to tune the model.

Development of Steering Quality Measures

In order to assess whether a particular run passed or failed the criterion, an evaluation instrument was needed which could be used to rate performance in abstract, objective terms. Absolute ratings of performance quality could then be assigned to transits by the Panamax vessel and validation ship, and the safety of the proposed layouts evaluated relative to the criterion performance. Toward this end, a multidimensional performance measure, referred to as the Steering Quality Profile (SQP), was developed for this study. The SQP consisted of four independent measures (or indices), each of which addressed a separate aspect of shiphandling. The four indices were developed as follows:

• Relative Clearance Margin (RCM) – Evaluates proximity to obstacles. This measure compares the minimum distance from a vessel to the traffic ship during a meeting with the minimum distance from the vessel to the bank. Based on the assumption (supported by the comments of PCC pilots) that the ideal strategy in a meeting is to split the available lane, the Relative Clearance Margin is designed to yield a perfect score of 1.0 when the ship-to-ship distance exactly equals the ship-to-bank distance, and the width of the maneuvering lane is minimized (i.e., no crab angle). Any deviations from this ideal case result in a reduced score, with the penalization for any given error becoming greater as the available lane becomes smaller, until a score of zero is awarded if the vessel strikes the bank or the traffic ship. The concepts involved in this measure are illustrated in Figure 4.

• Control Force Margin (CFM) – Evaluates control reserve. This factor was evaluated by comparing the amount of reserve control force (taking into account the rudder angle, velocity, engine RPM and tug forces) with the total force available throughout the meeting encounter. The perfect score of 1.0 is awarded if the meeting is accomplished without the application of any control effort, while a score of zero is obtained if the pilot expends all available resources.

• Reciprocal of Yaw Rate Variance (RYAW) and Course Changing Quality (CCQ) – Evaluate directional control in straight reaches and curves, respectively. In a straight reach, the yaw rate is sampled periodically during the meeting, and the variance of the sample points is computed. This quantity gives an indication of how smoothly the ship moves in reducing oscillations after the meeting encounter. The reciprocal of this value is then computed so that a higher score is indicative of better performance.

In a curve, the best mathematical "ideal" curve is fitted to the plot of the observed yaw rate time history. The residual error that cannot be accounted for by a smooth curve constitutes a measure of the degree to which the achieved course change deviates from the ideal case. For this measure, unlike the other measures discussed, a lower score indicates less error and hence better performance.

Establishment of Safety Baseline

To determine the specific safety levels against which proposed layout alternatives would be evaluated, the validation ship was modeled on the CAORF simulator. A computer autopilot module was developed to make shiphandling decisions in compressed time and tuned until its behavior resembled that of a sample of PCC pilots. This model was then used to carry out meeting encounters between two validation ships in each of the eight curves in the Gaillard Cut plus a straight reach. The Steering Quality Profiles generated in this way served as the safety criteria for each curve. A run performed by the Panamax ship which generated SQP values equal to or greater than the criterion level (or "baseline") was considered to have passed in that layout. The specifics of the scenario design are described in the following section.

Scenario Development

In order to ensure the accuracy and generalizability of the results of the compressed-time analysis, particular care was taken in the design of the scenarios to be performed. Two considerations were of primary importance: 1) creating scenarios that were realistic enough that the results of the compressed-time analysis would be useful when implemented in actual practice, and 2) creating situations that were difficult enough so that if passing performance were achieved under those circumstances, it could also reasonably be expected under most other circumstances in the real world. In general, different levels of the major study factors (i.e., operational conditions and layout alternatives) determined the exact specifications for the test scenarios. The meeting location was also determined to be of major importance, and was therefore considered as well. This section details the scenario construction for the compressed-time analysis.

Baseline Scenarios

The scenarios for which baseline safety levels were collected were conceptualized as "typical" of the situation now encountered by a PCC pilot in the Gaillard Cut. In other words, the objective of the widening project was to enable meetings between Panamax vessels to occur at the same level of safety as that now experienced with validation vessels under normal circumstances.

All baseline scenarios took place in the model of the present Cut; therefore, the layout factor was not of concern. The operational conditions had to be identified. This set of conditions, determined through discussions with PCC pilots, were as follows:

- Speed – 6 knots
- Cross-track starting position – on centerline
- Anticipation distance (to meeting point) – six shiplengths
- Tug use – not available

As stated, the intention in this project was to create a conservative solution by requiring passing performance in a particularly difficult meeting situation. The most suitable way of manipulating the difficulty level was felt to be via the location at which the meeting actually occurred. Toward this end, the opinions of several PCC pilots were solicited as to the location that would lead to the most difficulty in a meeting. Based on their comments, the scenarios were devised as follows:

- All meetings occur approximately one shiplength from the point of intersection of a turn (PI).
- When ownship is making a turn to the right, the most difficult situation involves meeting before the point of intersection. This is because the pilot can see the other ship sailing across his bow which may lead to the temptation to begin turning too early. In addition, the passing ship effects tend to pull the vessel to the left, interfering with the completion of the turn.
- When ownship is making a turn to the left, the most difficult situation involves meeting after the point of intersection. This is due to the fact that following the left-hand turn, the pilot will tend to oscillate about his desired heading for a short time. The presence of the other vessel reduces the width of the lane that the pilot has available for steadying up.

Once the meeting location was specified for a given curve, the initial position for the run was fixed at a spot seven shiplengths from the meeting point. This was done in order to allow sufficient opportunity for the vessel

ANTICIPATION DISTANCE		8 SL			12 SL		
	SPEED	4 KNOTS	5 KNOTS	6 KNOTS	4 KNOTS	5 KNOTS	6 KNOTS
CROSS-TRACK STARTING POSITION	BETWEEN BANK AND SAILING LINE	1	5	12	8	16	21
	SAILING LINE	2	6	14	10	18	22
	CENTERLINE	3	7	15	11	19	23
	BETWEEN CENTERLINE AND OPPOSITE SAILING LINE	4	9	17	13	20	24

TABLE 1. SUMMARY OF OPERATIONAL CONDITIONS
(NUMBERS IN CELLS INDICATE RELATIVE EFFECTIVENESS OF OPERATIONAL CONDITIONS, WHERE 1 IS BEST)

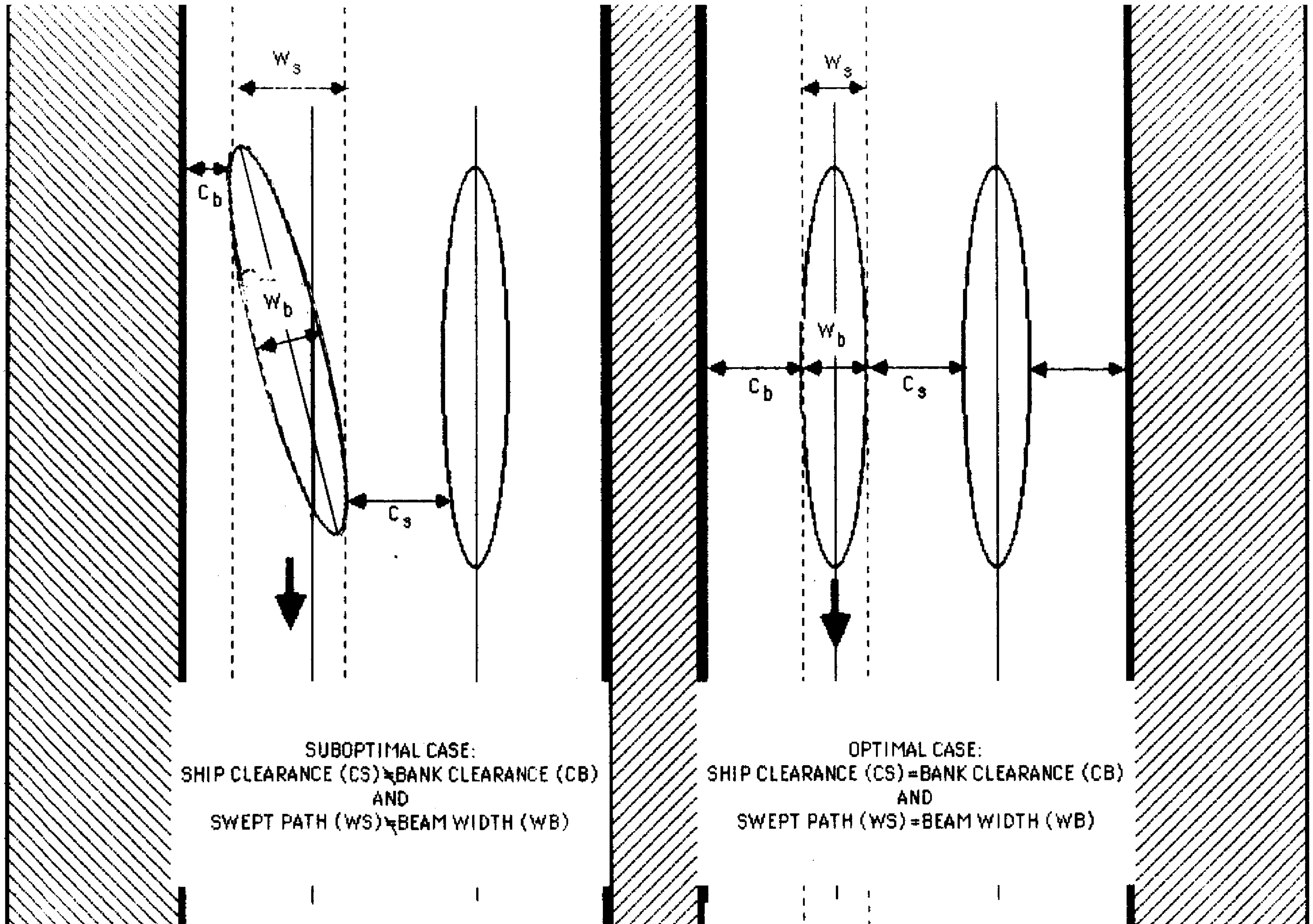

FIGURE 4. ILLUSTRATION OF PARAMETERS DEFINING RELATIVE CLEARANCE MARGIN (RCM)

to prepare for the encounter (e.g., steady on course, move to the sailing line, etc.). Since a layout must yield acceptable performance in both directions to be satisfactory, all scenarios were investigated for ownship moving in both directions. Therefore, a separate baseline value was collected for each of the ships in the meeting encounter.

Test Scenarios

In the scenarios performed to evaluate the alternative layout configurations, each of the major factors had to be considered in constructing the test scenarios. Since construction of the straight reach scenarios differed somewhat from construction of scenarios for the curve, the two will be considered separately.

Construction of Scenarios for the Straight Reach

Layout Conditions. The only layout factor manipulated in the straight reach was the channel width. In constructing the proposed layout alternatives, simplified geometrical representations of the channel width. In constructing the proposed layout alternatives, simplified geometrical representations of the channel layouts were used so that the data bases could be constructed via computer program. In the case of the straight reach, the channel was modeled as two parallel lines, and the width was defined as the distance between them.

Since every straight reach in the Cut would have been modeled the same way under this strategy, only one test was run and the results generalized to all areas of the Cut.

Beginning with the present 500-foot width, layouts were constructed in increments of 50 feet to a maximum of 750 feet. In order to maximize the comparability of the test runs and the baseline runs, the test scenarios were designed to be as similar as possible to those executed in the collection of the baseline data. Since the starting location was set at seven shiplengths from the meeting point, the straight reach was made approximately 14 shiplengths long. The layouts were rank-ordered by width with the narrowest as most desirable.

Operational Conditions. For the straight reaches, each of the 24 operational conditions described previously were tested at every width until the width was found which permitted passing performance at every operational condition. This data was required to provide the basis for rank ordering the operational conditions in the analysis of the curve sections. Furthermore, it served as an opportunity to observe the behavior of the compressed-time model in every situation which would be encountered during the project. Finally, since the straight reaches constitute the majority of the Gaillard Cut and are therefore extremely critical in determining the suitability of proposed improvements, this set of tests was important in that it provided conclusions which did not have to depend on any assumptions concerning the relative effectiveness of the various sets of operational conditions.

Meeting Location. Because the simplified straight reach was uniform in shape and width throughout, it was not deemed necessary that the meeting occur in any particular location. Therefore, the vessels were initialized 14 shiplengths apart and allowed to meet without restrictions or corrections.

Underkeel Clearance. One factor not previously mentioned which can significantly influence shiphandling is the depth of the water underkeel. Because the depth of the Gaillard Cut fluctuates seasonally, and the draft of transiting vessels can vary, the PCC expressed concern that the proposed layout be effective given changes in the underkeel clearance. To address this, the analysis of the straight section was run twice, once at a depth of 45 feet (5-foot underkeel clearance) and again at a depth of 50 feet (10-foot underkeel clearance). The results of these sets of tests were then compared to assess the generalizability of the proposed layout solution.

Construction of Test Scenarios for the Curved Section

Operational Conditions. The original intention of the analysis of the curved section was to test each of the operational conditions in progressively larger layouts until all had passed. As explained previously, this entailed organizing the operational conditions according to their effectiveness. The approach taken in this organization involved a statistical analysis of the behavior of the autopilot and the Panamax ship to determine (a) which of the individual operational variables (i.e., speed, cross-track position, and anticipation distance) are most closely related to performance, and (b) which levels of each of these variables led to the best and worst performances. The effectiveness of each operational condition was then determined by weighting the effectiveness rank (step b above) of the level of each variable within that condition by the importance of that variable (step a) and summing across the variables.

The arrangement of each individual variable obtained in this way was, from best to worst:

Speed – 4 knots, 5 knots, 5 knots

Anticipation distance – short, long

Cross-track starting position – between sailing line and bank, sailing line, centerline, between centerline and opposite sailing line

Upon surveying the results of this analysis, the PCC and CAORF staff recognized that the ordering of the anticipation distance was counterintuitive (i.e., it had been assumed that a short anticipation distance was an inferior strategy in general). However, a closer examination of the behavior of the autopilot shows these results to be reasonable. In moving from the centerline to the sailing line, the autopilot first calculates a point lying 75 percent of the distance between the point of the anticipation distance and the meeting point. A perpendicular is then dropped from this point to the sailing line, and the autopilot steers toward the sailing line at its intersection with the perpendicular. It was determined the autopilot would need to make a course change of approximately 1.6 degrees with a long anticipation distance, as opposed to 2.2 degrees with a short anticipation distance (see Figure 3). Although the ship would make a more abrupt course correction with the short anticipation distance, the difference was not great enough to cause the autopilot to have any more difficulty in steadying on the sailing line than it would using the long anticipation distance. In fact, the long anticipation distance would tend to bring the vessel into interaction with the bank forces sooner than would be the case with the short anticipation distance. Therefore, the long anticipation distance could be expected to yield slightly worse performance, as was the case in the results obtained in the compressed-time analysis of the straight section.

During the ordering process it was noted that the cross-track position variable accounted for only about 3 percent of the variance in the SQP measures, and was thus not of great importance in determining performance. This is not to say that cross-track position is not an important determinant of performance in general. Rather, because the vessels were initialized nearly one mile before the meeting location, they had sufficient time to recover from cross-track positioning error imposed upon them. The effects of this error, while not negligible, were thus dealt with by the autopilot rather easily in this study. Therefore, it was decided that the information gained by testing four levels of this variable did not justify the number of compressed-time runs required, and that only operational conditions including starting positions of centerline and sailing line would be tested. In addition, it was noted that one of the scenarios which had failed the straight section analysis at the 600-foot width comprised a 6-knot speed, a starting position to the right of the centerline, and a short anticipation distance. Under these circumstances, the autopilot apparently made an abrupt move toward the centerline at the outset of the run. Because of the short anticipation distance, it did not begin moving back to the sailing line until it had traveled almost to the centerline and, due to the higher speed, had acquired a good deal of lateral momentum. This momentum evidently led to increased shiphandling difficulty in making a smooth transition back to the sailing line, and thus reduced RCM. Keeping in mind that the short anticipation distance was included primarily as representing an individual piloting style, it was pointed out that a human pilot would most likely not return to the centerline from the sailing line in the presence of an oncoming traffic ship. Under these circumstances, a pilot who normally exhibits a short anticipation distance would probably choose to remain closer to the sailing line to prepare for the meeting. Since this behavior would be more closely approximated by the autopilot using the long anticipation distance, it was decided to drop the 6-knot, sailing-line starting position, short anticipation distance from the analysis. Thus, a total of 11 operational conditions were tested.

Layout Conditions. In the compressed-time analysis, each curve was treated as completely independent of other curves. That is, all layouts for a given curve were devised as if that curve existed in isolation, not taking into account its connections with other curves in the Cut. If every layout alternative for each curve had had to be investigated in combination with all layouts for both adjacent curves, the analysis would quickly have become impossible complex.

As in the straight section analysis, the layouts tested for each curve were simplified models specified only by the three main parameters (width, radius of curvature, and transition-zone slope). The straight sections on either side of the turn were represented as parallel lines. The outer bank was formed from the intersection of the straight reaches, at an angle equal to the deflection angle found in the present Cut. This outer bank remained constant for all layouts of a given curve.

The radium of curvature was operationalized as an arc with the specified radium, tangent to both centerlines of the adjacent straight reaches. To define the width of the turn (at width "w" for example), lines of length "w" were drawn from the outer bank through the points of tangency of the centerline arc. The inner bank was then modeled as an arc extending between the endpoints of the two lines described above, and concentric with the arc of the centerline. The radius of the arc of the inner bank was equal to the radius of curvature of the turn minus one-half the width, by this construction.

In those cases where the width of the turn was greater than the width of the straight reach, a transition zone was used. This was modeled as a pair of straight lines connecting the ends of the inner bank to the straight reach. The straight reach was then extended seven shiplengths from the point of intersection of the turn, since this was the distance at which the scenarios were designed to begin.

A computer program was devised to create data bases using every combination of the parameter levels as described previously. The PCC then computed the excavation volumes for a subset of these layouts, and used linear interpolation to estimate the volumes for the remainder. Using the excavation volumes to rank order the layouts according to desirability, the data bases were stored on computer disk to facilitate implementation of the search strategy.

Meeting Location. In general, the test scenarios were designed to be as close as possible to the baseline scenarios so that differences in the results could be attributed to the layout and not to the scenario particulars. Therefore, the meetings were programmed to occur as specified.

Starting Location. All test scenarios were constructed to begin with both vessels at seven shiplengths from the PI of the turn. As in the baseline scenarios, this was necessary to give the vessels sufficient time to prepare for the meeting encounter.

Procedure

For a given curve, the list of operational conditions was combined with the list of layout alternatives to form the problem matrix (see example, Appendix A). Beginning with the least costly layout, a meeting encounter was executed under the most effective operational conditions as outlined. Upon completion of the run, data was examined to ensure that the conditions under which the meeting actually occurred were as intended; specifically, the combined meeting speed had to be within plus or minus one knot of the designated speed, and the meeting location had to be within one-half shiplength of the intended location. If these tolerance limits were not met, the program automatically adjusted the initial speed and/or starting location and performed the run again. When a run was completed within the limits, the SQP values were computed and compared with the baseline. If the run passed the criterion, then the next set of operational conditions were selected; if it failed, the next layout was loaded according to the decision strategy. The entire procedure was designed so that all runs for a given curve could be executed and analyzed without the need for human intervention.

Conclusion

The compressed-time analysis was successfully executed; however, the results are currently under study and are not available for publication at this time. However, several observations can be made which testify to the validity of the compressed-time methodology for channel design. First, the dimensions recommended via this study have been evaluated against guidelines prepared by the Permanent International Association of Navigation Congresses. Two important parameters described in these manuals as influencing safety are the radius of curvature as a function of the vessel length, and the channel width as a function of vessel beam. The widths determined in this study generally match the design manual recommendations, given a ship with the beam of the Panamax vessel. However, in two particularly sharp bends in the Cut, the margin of safety which was not considered by the design manuals. The ratio of radius of curvature to shiplength recommended via the compressed-time analysis were substantially greater than those recommended by the design manuals for curves of comparable sharpness. However, the radius of the curves in the existing Cut are also greater than those recommended when the validation vessel is used as the comparison. The radius to shiplength ratio for Panamax vessels in the recommended improvements is similar to the ratio for validation vessels in the existing Cut; thus, the safety level of the existing Cut is maintained as per the objectives of the study. Also, a visual model of the proposed layouts was constructed on the man-in-the-loop simulator, and a sample of PCC pilots carried out meeting encounters between Panamax vessels. In seven of the areas tested, the average performance of the pilots aboard the Panamax vessels equaled or exceeded the baseline level of safety. In only one area were minor improvements needed to bring the safety level up to the criterion.

The utilization of simulation for testing and validating proposed waterway layouts, especially under extreme environmental conditions or emergency situations has gained wide acceptance in the maritime community.

This study has demonstrated that simulation can also serve as a cost-effective tool in the design process as well. By using the compressed-time methodology to screen the layout alternatives and eliminate those that are unacceptable, the more costly real-time visual simulator with man-in-the-loop can be most efficiently applied to the final confirmation process. This outcome should therefore serve to encourage the use of simulation as an aid in future port and waterway design projects.

Appendix A

Illustration of the Compressed-Time Procedure

In this section, the proposed methodology is applied to a simple selection process as an illustration of the Compressed-Time Analysis Strategy.

Application of the Basic Strategy – In order to visualize the problem, a matrix of all possible combinations of channel layout and operational conditions is created. The vertical axis represents the various sets of operational conditions arranged in the order that they will be examined by the Compressed-Time Analysis Program. The horizontal axis is a similar representation of the different channel layout alternatives. As described previously, the operational conditions will be ordered according to their effectiveness (i.e., the quality of the performance that can be expected from each one) and the layout alternatives will be ordered according to how much dredging each one will require. Figure B1 presents a hypothetical matrix with six levels each of operational condition and layout alternative. The most effective operational condition is at the bottom of the vertical axis, and the layout requiring the least dredging is leftmost on the horizontal axis. If each combination of layout and operational condition is considered as a "scenario," then the scenarios are represented by the numbers within the cells of the matrix.

In this analysis, two assumptions are made. First, it is assumed that the operational conditions will be ordered correctly in terms of effectiveness through the use of empirical testing and that this ordering will be consistent for each curve. Second, it is assumed that as more dredging is accomplished for a layout, ship performance will improve. If these assumptions are exactly correct, then it should be the case that for the matrix in Figure B1, scenarios that will yield passing SQI's will cluster in the lower right corner (more effective conditions and layouts with more dredging) and inversely, scenarios yielding failing SQP will cluster in the upper left corner. The +'s and -'s in the cells of the matrix denote passing and failing performance respectively for this hypothetical example. Notice that, in the ideal case, there is a distinct boundary between passing and failing scenarios. This boundary represents the least costly dredging alternatives that will permit safe meeting while imposing the least restriction on operating conditions.

Applying the Compressed-Time Analysis Strategy to the problem, the program begins with the most effective operating conditions and least-dredging layout alternative, or Scenario 1. This scenario yields passing performance so these conditions are recorded (denoted by the circle around the + on the matrix) and the program selects the next less effective set of operational conditions. This represents a move upward on the matrix, and leads to Scenario 2 (as indicated by the arrow). This scenario fails so the program selects the layout requiring the next greater amount of dredging, represented by a move to the right on the matrix leading to Scenario 8. This scenario fails as well, so the program moves right again, to Scenario 14. This passes, so the conditions are recorded and the program moves upward to Scenario 15. This also passes, and is recorded, and the program moves upward again. Scenario 28 passes, the conditions are recorded and the program moves upward. Scenario 29 fails so the program moves right to Scenario 35. This passes and is recorded and the program moves up to Scenario 36. This passes and, because this is the least effective set of operation conditions, the program terminates. In this way, by creating the matrix and remembering two rules [i.e., (1) passing performance leads to a move upward and (2) failing performance leads to a move to the right] it can be seen how, for each operation condition, the program examines only enough layouts necessary to identify the one which permits safe meeting with a minimum of dredging. In this example, only one-fourth of the possible scenarios had to be examined in order to identify the most efficient alternatives.

Compensation for Ordering Errors – As noted, a drawback to the basic strategy is the consequences of an improper ordering of the effectiveness of the sets of operational conditions. For example, if the fourth most effective set of operational conditions in Figure B1 is mistakenly determined to be the second most effective, the scenarios would be ordered as in Figure B2. Notice that the row associated with the fourth most effective set of operational conditions has been moved to the second spot, this represents the order in which the scenarios will be tested. The outcomes associated with each individual scenario remain the same as in Figure B1, reflecting the actual degree of operational effectiveness for this set of conditions. Beginning with Scenario 1, the program moves upward and immediately encounters a failure in Scenario 4. Moving to the right, failures are encountered in Scenarios 10, 16 and 22 because Operational Condition 4 is in reality much less effective than Condition 2. Scenario 28, 26 and 27 pass and are recorded, 29 fails, 35 passes and the program terminates at Scenario 36. The resulting difficulty is that Scenario 26 and 27 are identified as the most efficient for Operational Conditions 2 and 3, whereas Scenarios 14 and 15 are the more efficient passing alternatives. Due to the misordering of the operational conditions, these scenarios are never tested.

In order to overcome this difficulty, a check is built into the program whereby, whenever a failing scenario is encountered, the next less effective set of operational conditions is examined for that layout. If this scenario also fails, the program proceeds. However, if it passes, an error is suspected and the operational conditions are rearranged so that the passing scenario will be lower in the test order than the failing scenario. In Figure B3a, the misordered problem discussed in Figure B2 is presented again. The basic program proceeds from Scenario 1 to Scenario 4. Here, since Scenario 4 fails, the next less effective set of operational conditions is tested, represented by a move upward into Scenario 2 (open arrow). Scenario 2 fails also, so the program returns to Scenario 4 and proceeds in the normal manner to Scenario 10. This fails, so Scenario 8 is checked and fails. The program then returns to Scenario 10 and proceeds to Scenario 16. Since error is suspected and the operational conditions associated with Scenarios 16 and 14 are reversed. Figure B3b shows the results of this reversal. Once the new order is established, the program proceeds. Scenario 14 was the last tested and passed, so the program moves upward from there. Scenario 16 fails so the check is performed, moving up to Scenario 15. This passes so an error is suspected. The levels of the operational conditions associated with Scenarios 15 and 16 are reversed, resulting in Figure B3c.

From here it can be seen that the program will proceed in a normal fashion to the end. Quickly, Scenario 15 passes so the program moves up to 16. Sixteen and 22 fail, 28 passes and is recorded, 21 fails and 35 and 36 each pass and are recorded. Thus, using this procedure, the same cost effective scenarios are identified as in the ideal case.

Sensitivity Analysis – Another possible source of error in the Compressed-Time Analysis is that under some isolated circumstances a layout alternative requiring less dredging might lead to a better performance than an alternative requiring more. Due to the nature of the basic procedure, this type of scenario would probably not be identified. In Figure B4a, the layout associated with Scenario 17 yields better performance than the layout associated with Scenario 23, but is not tested. Since Scenario 17 requires less dredging, it would represent a better solution than Scenario 23. The sensitivity analysis is run after the initial analysis is complete. It simply identifies the passing scenarios selected by the basic procedure and, for each one, tests scenarios associated with the three layout alternatives requiring less dredging. Thus, an "error envelope" is built into the analysis as demonstrated in Figure B4b. Here, Scenario 17 is identified and selected for possible on-line testing.

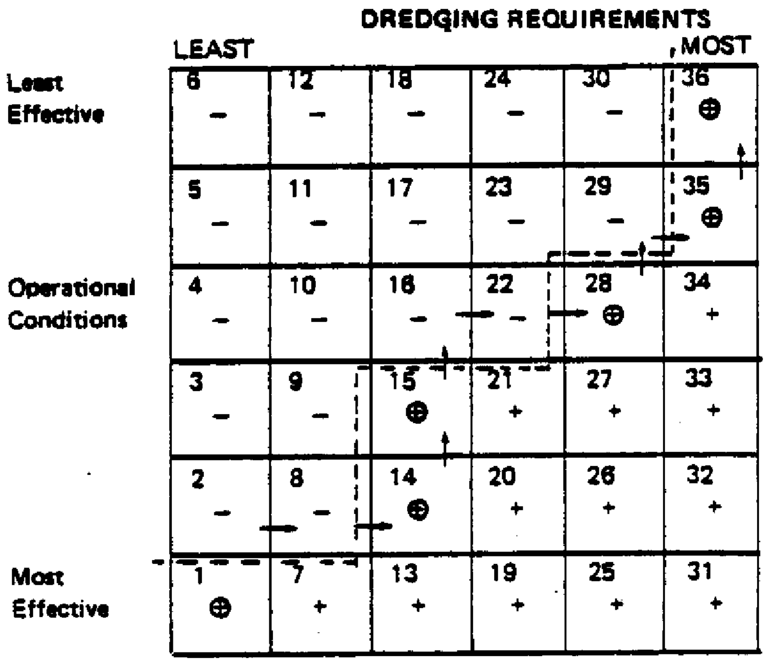

Figure A1

—► Direction of Scenario Selection

\+ Passing Scenario

− Failing Scenario

— Boundary Between Passing and Failing Matrix Areas (Ideal Case)

O Scenario Selected for On-Line Testing

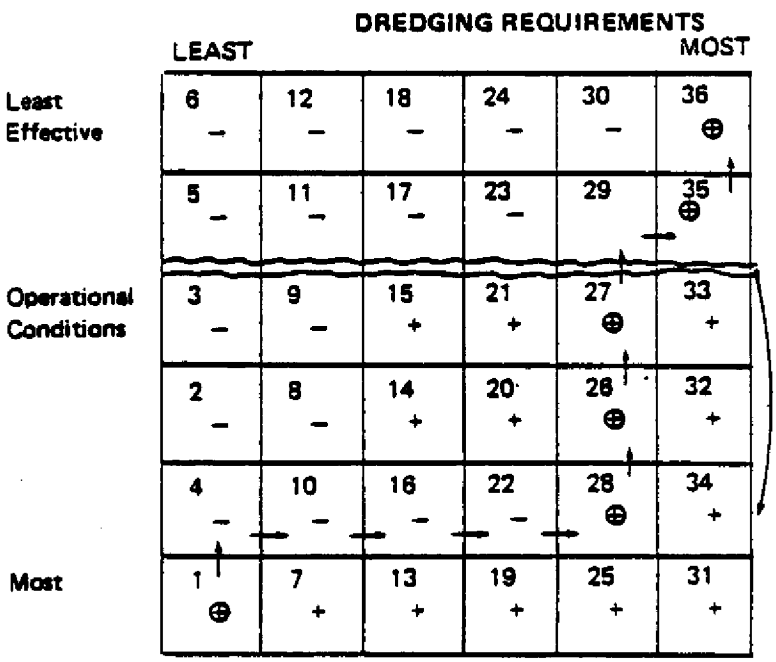

Figure A2

Arrow indicates erroneous placement of Condition 4 in the testing order

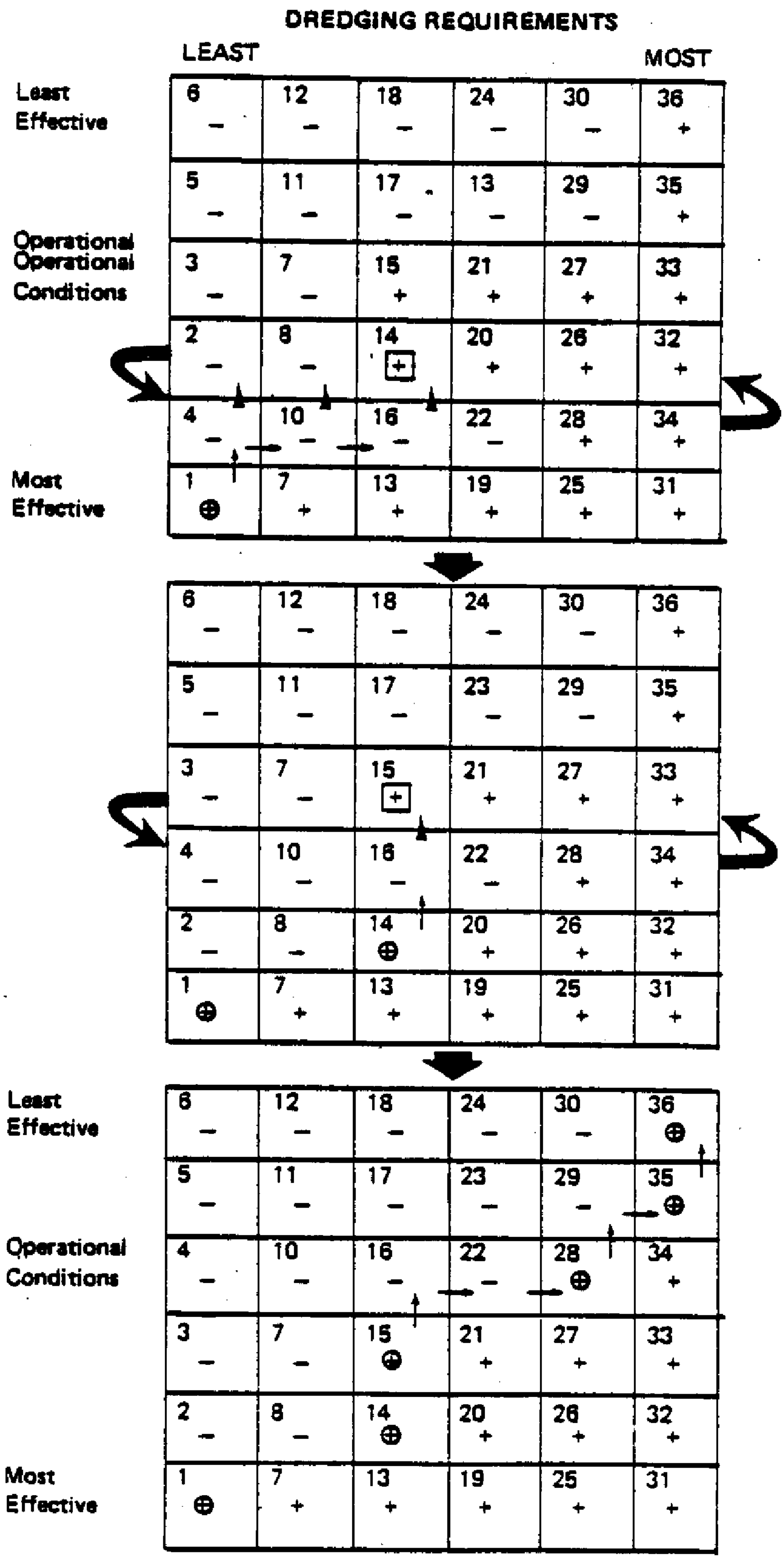

Figure A3a

—— Direction of Testing

Direction of Checking for Effectiveness Ordering

☐ Identification of Scenario Violating Checking Assumption

Arrows indicate reversal in order of levels of effectiveness associated with scenarios violating checking assumption.

Figure B3b

Figure A3c

DREDGING REQUIREMENTS

Figure A4a

	LEAST					MOST
Least Effective	6 −	12 −	18 −	24 −	30 −	36 ⊕
	5 −	11 −	17 $\boxed{+}$	23 −	29 ⊕	35 +
Operational Conditions	4 −	10 −	16 −	22 −	28 ⊕	34 +
	3 −	9 −	15 −	21 ⊕	27 +	33 +
	2 −	8 ⊕	14 +	20 +	26 +	32 +
Most Effective	1 ⊕	7 +	13 +	19 +	25 +	31 +

Figure A4b

	LEAST					MOST
Least Effective	6 −	12 −	18 −	24 −	30 −	36 ⊕
	5 −	11 −	17 +	23 −	29 +	35 +
	4 −	10 −	16 −	22 −	28 +	34 +
Operational Conditions	3 −	9 −	15 −	21 +	25 +	33 +
	2 −	8 ⊕	14 +	20 +	26 +	32 +
Most Effective	1 ⊕	2 +	13 +	19 +	25 +	31 +

Perspectives of an A&E Firm on the Cost Sharing Legislation

Richard F. Thomas
Gahagan & Bryant Associates

Introduction

The pending cost sharing legislation presents a significant business opportunity for consulting firms experienced in dredging engineering. Like all good business opportunities it is a mutual benefit: a client for the consultant and services of value to the client.

Why does a port need A/E services?

(a) if it doesn't have in-house skill and experience;

(b) it simply doesn't have the available people to get the job done;

(c) if if wants an independent review of its programs.

Historically, the federal government through the U.S. Army Corps of Engineers has carried out all channel and harbor improvements on authorized navigable waterways. Projects were authorized and funded by the Congress through the omnibus water projects bill. As of October 1, no Omnibus Bill has been passed since 1978. As is well appreciated by this group, this situation is the result of the objective of past administrations to require participation in the cost of water projects by local beneficiaries.

In addition to the obvious budget balancing aspects, many would agree that a requirement for cost sharing will tend to improve the economic efficiency of water and navigation projects. In any case it appears that we are in a revolution in the methods used to finance navigation projects.

In addition to project financing needs, local port authorities are likely to have a much different perspective regarding the design and construction and maintenance costs of a channel project when they pay a significant share of the cost of the project. It would seem that it is not appropriate for a local agency to rely entirely on the federal government, i.e. the Corps of Engineers, for technical and cost analyses when that local agency is responsible for the expenditure of millions of dollars of state and local funds for a dredging project.

The sharing of costs has two significant effects:

(1) Heightened local interest in the project cost.

(2) Formation of a stronger local/Corps partnership.

We believe that the significantly increased local costs resulting from the pending legislation presents an important opportunity for consulting firms specializing in dredging engineering. This opportunity lies in the increased pressure being placed on the ports to improve cost efficiency at a time when their share of dredging costs are dramatically increasing.

We believe that the ports need their own technical support. This is not seen as an adversarial relationship. The Corps may have different directions, different interests and rules which may not always fit in with the port directions and interests. A port may have a much more complicated requirement for development than just dredging its channels. Development of new land with dredged material is an obvious example.

Development of new land area with dredged material may require careful scheduling of a dredging project in order that the best material is placed in the development area. The practicality and costs of such an action may be an important matter for discussion with the Corps. What are the feasibility and cost of changed handling and scheduling needs? The port will need the technical expertise to carry on this discussion. Consultant, A/E involvement may be of great benefit to the port.

This paper will discuss some engineering factors involved in cost sharing for channel projects rather than the political aspects of the pending cost sharing legislation.

Factors Involved

In this paper we will concentrate on a narrow aspect of harbor dredging projects. It is appropriate, however, to outline the broader issues involved.

The analysis of public works projects can be based upon four principal factors: Engineering, Economics, Institutions and Environment. These factors and selected elements involved in each are indicated in Table 1. It is felt that there is ample proof for the statement that no significant project can be successful without full consideration of all of these factors and their interrelationships. The process by which this is achieved is also critical.

Table 1.
Factors in Port Dredging Projects

ENGINEERING	Navigation Needs Safety Hydrographic Surveys Material Characteristics Dredging Equipment Capabilities Disposal Areas Relocations
ECONOMIC	Dredging Costs Disposal Area Costs Project Benefits Project Financing
INSTITUTIONAL	Public Interests Commercial Interests Local Government State Government Federal Government State Agencies Federal Agencies
ENVIRONMENTAL	Ecosystem Conditions Project Effects Project Permits

Three Examples

The impact of cost sharing as well as the traditional cost responsibilities of the local sponsor on dredging project efficiency can be indicated by three examples:

50-foot Project, Baltimore, Maryland

Project redesign and stockpiling of dredged material.

Pelican Island Disposal Area, Galveston, Texas

(1) The State of Maryland had an economic analysis prepared of the benefits resulting to the state from the 50-Foot Project. Or put in another way, how much investment by the state was warranted? Without getting into the details and the actual amounts involved, the state determined that they could not afford their share of the cost of the project as presented. Some redesign resulting in a reduced cost project was desirable. Fourteen alternative channel arrangements and their cost were evaluated. This is an example of the situation where Corps B/C analysis and project formulation may have a somewhat different approach or objective than the local port.

Working with the Corps, the pilots and other interested parties, some changes were made in the project dimensions and the project cost was reduced to a level that the state felt it could handle.

(2) A second example, also with the 50-foot Project, is the interest of the Maryland Port Administration in stockpiling the sands and firm clay that will be dredged for use in the temporary raising of the perimeter dike at Hart Miller Island. This dike raising may be required in order to fully utilize the capacity of the site.

Approximately 2.6 million cy of suitable materials is available out of a total of 27.8 million to be placed at Hart Miller Island as part of the 50-foot Project.

Stockpiling involves questions of the scheduling of work, the additional cost, if any, of placing the material in a stockpile and the resulting value of the stockpiled material to MPA.

Pelican Island Disposal Area

In 1983, the wharves embarked upon a Disposal Area Management Plan (DAMP) at the Pelican Island

Disposal Area that includes the drying and shrinkage of dredged material in the disposal area as well as dike and spillway raising. The crust management program not only gains capacity but also makes the material better for dike raising. This represents a significant saving over importing fill material which was required by past practices.

The DAMP program assures that the maximum life of the disposal area will be realized by maximizing the volume of dredged material contained at the site consistent with foundation stability.

The concept of crust management is based upon the techniques described in the reports of the Dredged Material Research Program of the Corps Waterways Experiment Station (Reference 1).

The crust management program is carried out principally by the construction and maintenance of a network of shallow trenches to encourage rapid runoff of precipitation which allows evaporative drying of the dredged material to take place. Depending upon specific site conditions, shrinkage of up to 50 percent may be achieved.

Disposal area maintenance costs are summarized in Table 2.

Table 2
Disposal Area Costs

Period	Capacity Gained, cy	Cost	Cost per cy
1984 - 1985	5,100,000	$870,000	0.17
1985 - 1987 (1)	6,000,000	970,000	0.16
1985 - 1987 (2)	6,000,000	670,000	0.11

(1) if dike raising is necessary
(2) if dike raising is not necessary

In closing I would like to say it's pretty hard to come up with a new idea. The conditions and approaches described have always been relevant to the relationship between the Corps and the local ports. The new legislation simply increases the sensitivity of the port to their costs, their opportunities and their efficiency. This can only be viewed as positive.

Reference

Guidelines for Dewatering/Densifying Confined Dredged Material, Dredged Material Research Program, Technical Report DS-78-11, Waterways Experiment Station, September 1978.

Biodata

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The Capability of the Pneuma Pump for Continuous High Solids Transport

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Abstract

The potential of the Pneuma pump has intrigued European, American and Japanese dredgers. The potential has not been realized in practical dredging operations. This paper presents the reasons for the past erratic behavior. It also presents a resolution of the technical problems. This paper also presents the limitations of the Pneuma pumping system so that experienced dredgers can decide where its particular capabilities can be used. Since most of the dredging equipment in the United States is underused, the Pneuma pump must fill a specific niche or it will not be used. Finally, the aim of this paper is to elucidate the inherent capabilities of the Pneuma system based on slurry transport principles.

Continuous High Solids Pumping

Introduction

The existing technology for pumping solids varies widely and tends to be specific to each unique application. The effectiveness of such pumping systems is measured by the ratio of solids transported for a designated distance, together with the minimization of side effects such as turbidity. The success of these systems has been somewhat diverse, as measured by the percent of solids conveyed in a slurry. For example, non-industrial applications are generally in the five percent to 15 percent range, coal and mineral slurries up to 50 percent, and commercial dredging usually less than 14 percent. The higher ratio indicated for coal and mineral slurries is achieved through various pre-pumping comminution techniques.

The economics of dredging is predicated upon solids removed from the dredge site and disposed at some alternative site. The transporting of water is expensive and must be minimized for optimum performance. Generally two dredging alternatives exist for the effective transport of solids in a slurry, namely (1) centrifugal pumps where high volume of solids can be achieved by utilizing large pump units or (2) an air-driven positive displacement pump capable of handling high solids ratios, large particle sizes and long transport distances. There are costs, energy usage and handling considerations which must be evaluated for each approach.

Current media focus has been directed towards the removal of toxic materials from waterways. The removal of pollutants residing in waterways is delicate for two reasons – the innate problems associated with pumping turbidity (which may further exacerbate the situation), and the problems associated with treating the discharge water which is a by-product of the procedures.

This paper is basically a discussion of a method to improve the slurry pumping capacity of a pneumatic slurry pumping system. Thus, it is worthwhile to establish the parameters of the current system. The definition of centrifugal pumped slurry systems will also be discussed for comparative purposes. Figures 1 and 1a are a schematic presentation of the pneuma system. Figure 1 shows a version of the pneuma pump where the main tank is divided into three segments (A, B and C). Another version of the basic design is to incorporate three separate tanks which are joined by headers. Figure 1a is a layout of the system on a plane for the purpose of simplifying the visualization of air and slurry flows.

Segment A (Figure 1a) is in a condition of incipient discharge. Thus, the slurry is stationary; the air valve is closed to flow both to the vent and to the compressed air source; the slurry inlet valve is maintained in a closed position by the combined head of the slurry and the air pressure in the space above the slurry; the ball check in the slurry discharge header is maintained in a closed position by the discharge pressure in the slurry discharge pipe. In segment B, the air valve is open to the compressed air source and the air pressure maintains the slurry inlet valve in a closed position and drives the slurry through the slurry discharge pipe. In segment C, the air valve is open to vent (where the vent pressure usually is atmospheric but can be a chose pressure such as sub-atmospheric).

Figure 2 is a slightly idealized schematic of the slurry discharging and filling systems. When the slurry in segment A is discharged, the air valves are switched so that the air in segment A is vented and the air pressure source to segment B is switched on. The sequence continues as shown in Figure 2. Level controls in each segment define Segment "Full" and "Empty."

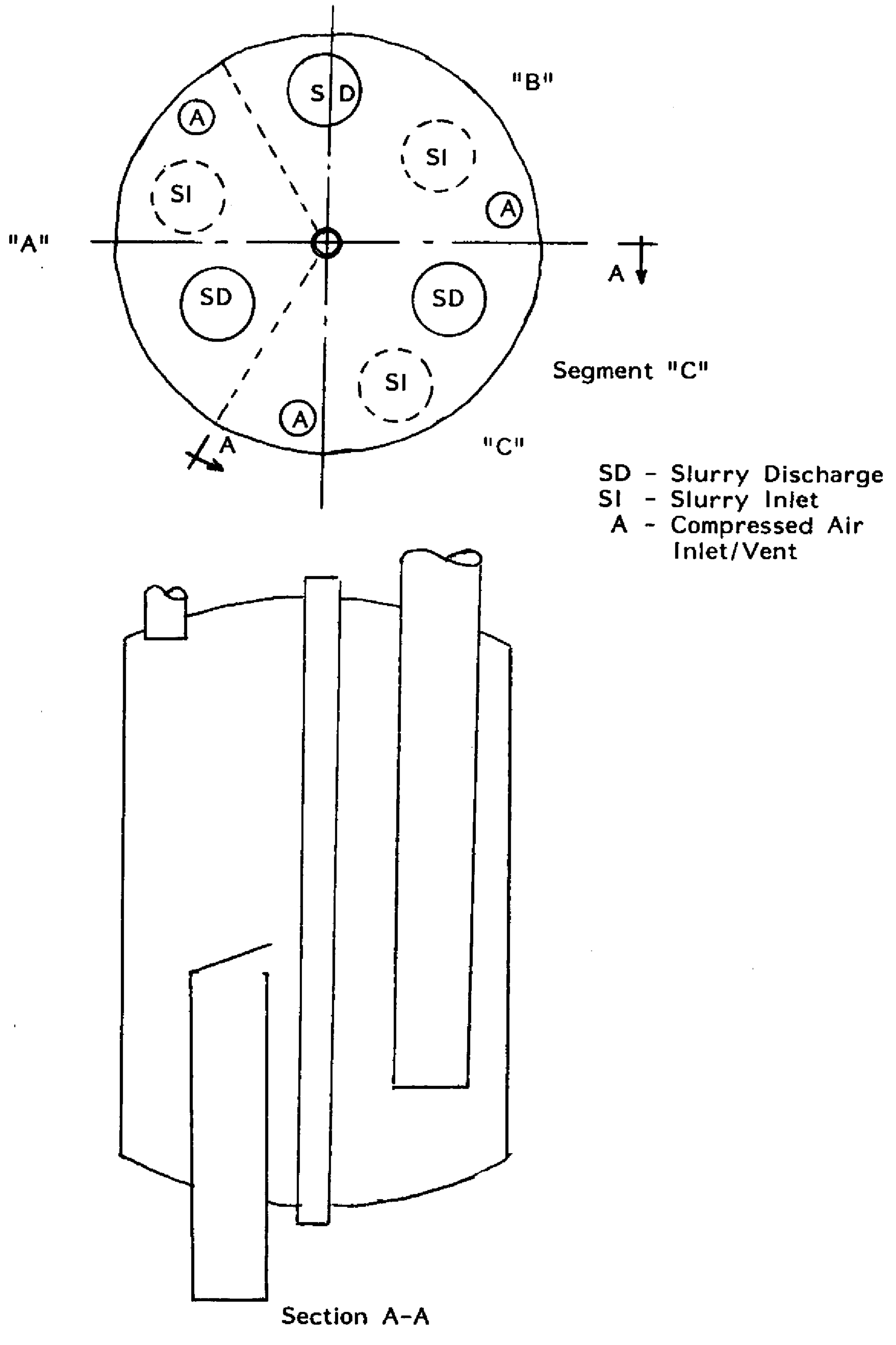


Figure 1. Pneuma Pump Schematic

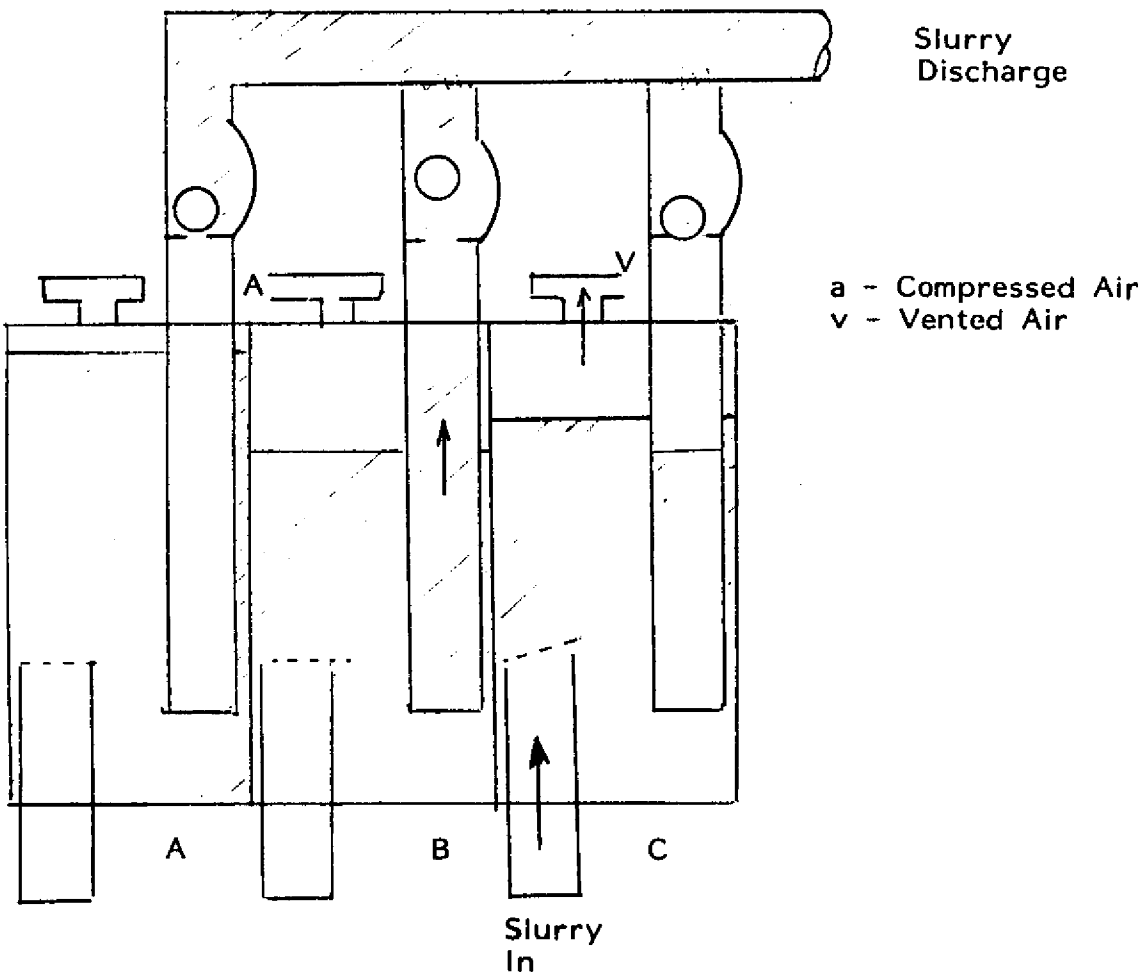


Figure 1a. Planar Representation of Segment Status

Pneumatic Pumping Examples

Two examples of pneuma pump operation are the Aqueduct Restoration and Reservoir Dredging (Courtney and Jones, 1983). In both cases, the three segment design was used. In the aqueduct restoration, the three segments were contained in one tank 5.5 feet in diameter. In the reservoir dredging, three separate 6-foot-diameter tanks comprised the three segments.

During the aqueduct restoration, dredging usually occurred at a depth of 30 feet (see Figure 3). The 10-inch (8.75 in. ID) polyethylene discharge pipe was surface-floated, and it was 6000 feet long. The discharge pipe raised 14 feet over a berm before discharge into the containment area. The pneuma pump was mounted on a tracked carrier. The intake of the carrier had a grid facing (3 in. by 4 in. openings) that plowed into the soil banks to be dredged.

The shore-based operator drove the intake scoops into the banks. When engagement was good, the slurry volume ratio was high ca 50 percent; if the carrier moves too slowly through the thin sediment layer, the slurry percent solids will be low. In normal use, slurry percent solids range from 20 percent to 50 percent solids (Courtney and Jones, 1983).

In the reservoir dredging, the dredging depth was approximately 50 feet. The containment area was over 500 feet above lake level and over 2500 feet from the Pneuma pump. A grid (3 in. by 6 in. openings) was a component of the spoil plow. This grid was pulled through the sediment. When engagement with the sediment was "good" and the correct amount of water was supplied, a slurry of 40 percent solids was regularly delivered to the containment area. When that engagement was not good, then the percent of solids in the slurry fell off.

Note that in both of the systems described above, a high solids ratio slurry was pumped intermittently. The reasons for the intermittent pump lies in solids engagement. If the sediment was in the correct lay for adequate solids capture, the slurry pumping operation was quite efficient; however, if either the lay of the sediment, or the ability of the operator to control the advance of the plow or scoop, deteriorated, then the percent solids ratio of the slurry decreased.

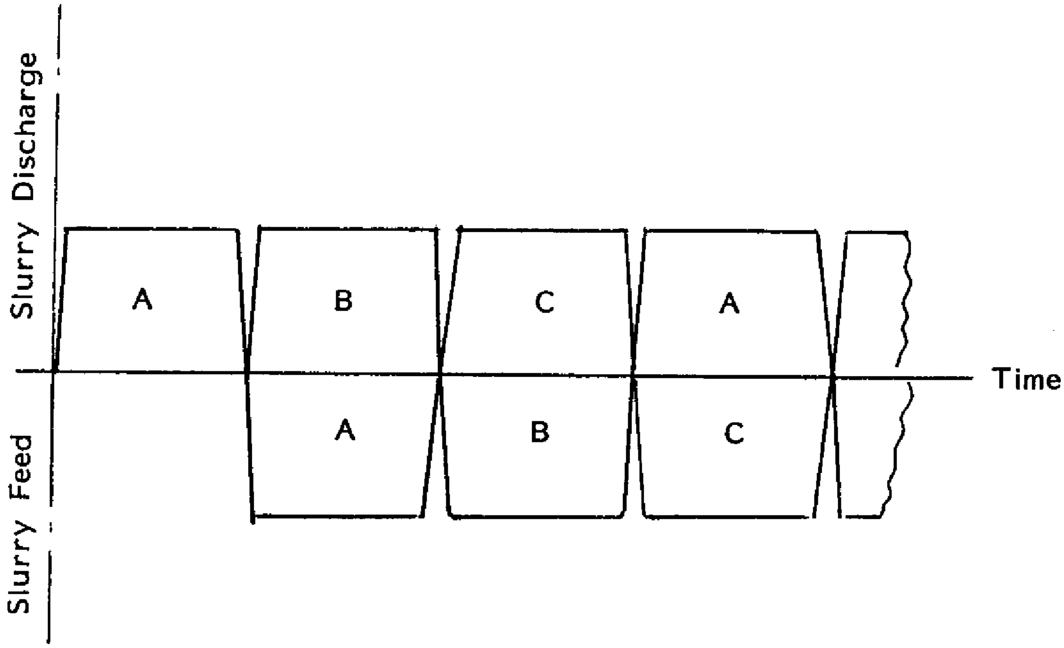


Figure 2. Segment Pumping Sequence

Comparison of Pneuma Centrifugal Pumps

An evaluation of the capabilities of the pneuma pump compared to the capabilities of the centrifugal pump is provided in the following table. The last entry in each comparative rank contains a paragraph number. A paragraph by number follows the table presenting amplification where it is required.

Before launching into a detailed discussion of the pneuma and centrifugal pump qualities, it is worthwhile to present a few definitions. The percent solids can refer to either weight or volume. Figure 3 shows the effect of percent solids (volume and weight) for a particular slurry; where the liquid is water and the solid has a specific gravity of 2.65. It would probably lead to less confusion to specify the specific gravity of the slurry rather than percent solids.

Economics of Dredging

The following discussion presents a sample dredging case. The effects of a low solids ratio are noted. The capabilities of pneuma centrifugal pumps are presented for different dredging depths, total lift head and discharge pipe length. Finally, the energy requirements for both pumping methods are evaluated.

This discussion considers the benefits of a high solids ratio dredging; the effect of shallow depth on dredging problems; and the benthic layer concerns. A particular case will be used for purposes of illustration. It will be assumed that the dredging operation is to deepen a channel six feet for a width of 15 feet and for a length of 300 feet. This will require the removal of 1000 yards of solid. Table 1 is a comparison of dredging effects for a small, medium and large dredge. The base of comparison is a slurry flow rate of 2 cubic feet per second for the small dredge, 10 cubic feet per second and 50 cubic feet per second for the medium and large dredges, respectively. The solids to water volume ratios range from 15 to 50 percent.

Unstated in Tables 2 and 3 is the assumption that a pump and a slurry transport pump exist. The mean flow rate (u) of a slurry is usually not less than 6.0 ft/s even for a fine, soft, well-dispersed silt. If the slurry consists of high density large particles, the mean velocity (u) will usually be less than 20 ft/s. For the slurry pumping rates of 2, 10 and 50 ft^3/s the slurry transport pipe size diameters will range from 8 to 5; 18 to 11; and 40 to 25 inches respectively, where the smaller diameter is associated with the higher velocity. The higher that the mean flow velocity is, then the greater the wear rate and the power requirements will be.

Consider that the volume of water required to dredge 1000 yards of solid is independent of the size of the

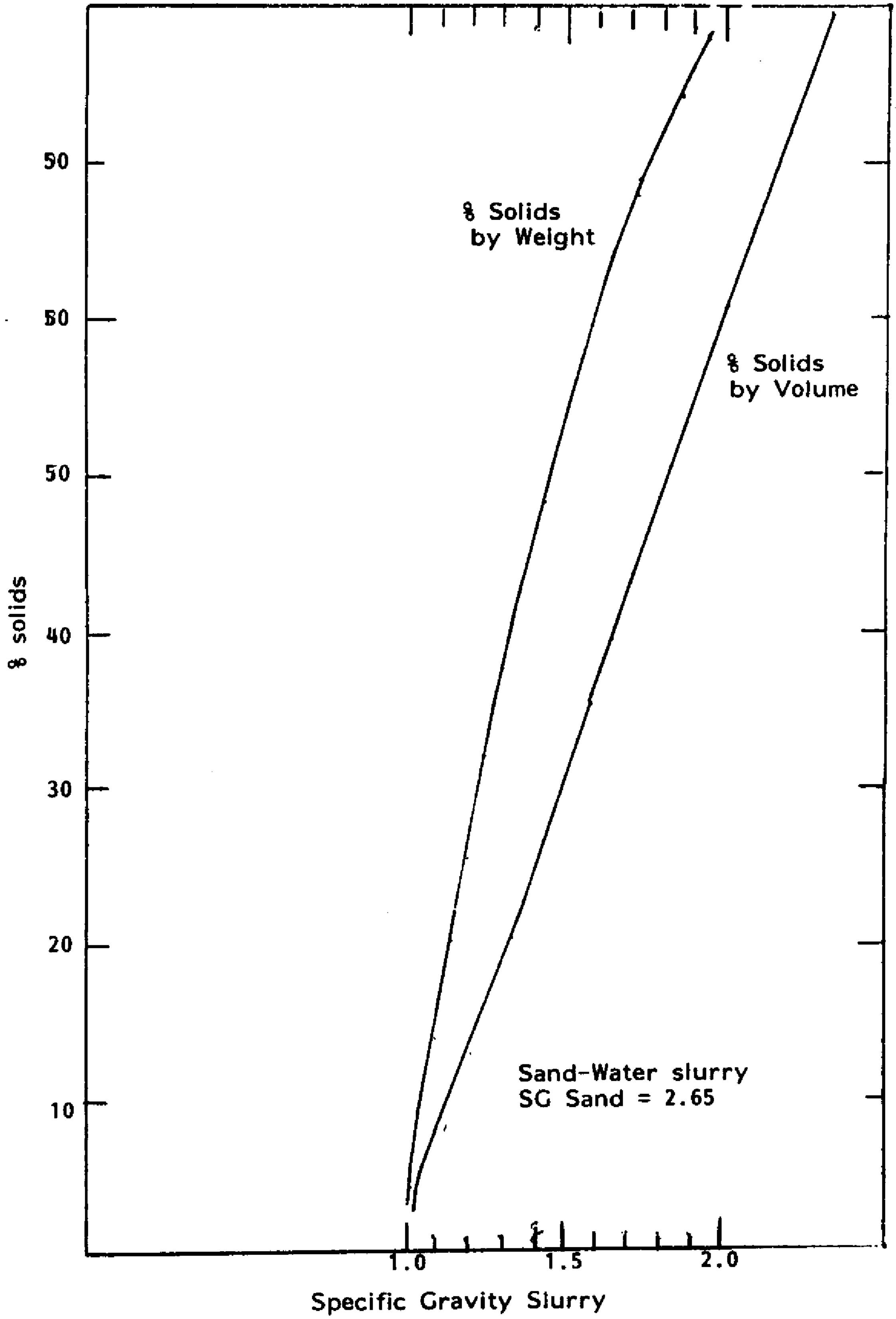

50
50
50
40
30
20
10
% solids
% Solids
by Weight
% Solids
by Volume
Sand-Water slurry
SG Sand = 2.65
1.0
1.5
2.0
Specific Gravity Slurry

Figure 3.

dredge for a given solids ratio. Table 2 shows the time lapse required to dredge the 1000 yards of solid. The inherent benefits of high solids ratio slurry transport are quite obvious. The choice among small, medium and large dredges is an economic one based on set-up costs – at both ends of the slurry transport pipe.

Note that if the run-off water must be treated, a high flow rate of slurry together with a low percent solids combination could be difficult to handle. As a point of interest, Table 4 shows the time required to dredge the 1000 yards of solid for using a small, medium or large-sized dredge, and the table also shows the volume of water that must be disposed of at the discharge end.

The size of the compressor and the power required are of interest to the dredger. Table 4 presents these requirements for a number of dredging requirements so that one will understand the limits of application. The centrifugal pump power and the total head are also listed for comparative purposes. The lower limit of the mean speed (u) of the slurry flow rate is chosen as 6 ft/s with an upper limit of 15 ft/s. The inner diameter of the discharge pipe is shown. The pump power levels shown in the table are theoretical values required to transport the slurry through the discharge pipe. A typical centrifugal pump will operate with an efficiency of 65 percent. The compressor can operate at efficiencies up to 90 percent; however, that efficiency would only occur if the compressor chosen closely matched the air flow and pressure head required by the pneuma pump. The total pressure head is presented in both feet of water head and psi for convenience.

For the sake of completeness, the requirements for a slurry rate of 50 ft^3/s are presented here in the body of the paper. To transport a 5-percent solids slurry through the 39-inch diameter pipe at a mean flow rate of 6 ft/s, a 4400 scfm compressor with a 134-hp motor would be required; and to transport a 30 percent solid ratio slurry at 15 ft/s through a 25-inch pipe, a 6000 scfm compressor with a 450-hp motor would be required.

Note that in Table 4, the specific gravity of the slurry (P) is assumed to be constant for each entry. Particularly, if one wishes to transport a high percent solids slurry then one must use a mass flowmeter to maintain that high percent. If the percent solids is allowed to decrease the loss of solids transported causes an economic loss. If the percent solids is allowed to increase – without control – the threat of a plugged discharge pipe must be considered.

Table 1. Comparison of Pneuma and Centrifugal Pumps

Item	Description	Pneuma Pump	Centrifugal Pump	Paragraph #
A	Percent solids transported through discharge pipeline	Vol. 50%	Vol. 15%	1
B1	Head	p 1000 ft	350 ft.	2
B2	Capacity	10,000 gpm	40,000 gpm	
C1	Effect of fine particles	Same	Same	3
C2	Effect of large particles	Limited by inlet and discharge pipe size	Pump clearance	
D	Turbidity development	Excavator dependent	Excavator dependent	4
G	Energy requirement	High	Low	6
H	System control	Same	Same	7
I	Total Cost	Variable	Less variable	8
F	West of components	Low	High	5

(1) The volume fraction of 15 percent for centrifugal was taken from a study (Interthal et al., 1983) in flow enhancement using a high polymer additive. The solid was 225 to 250 μm sand. For the same pressure drop (0.28 bar/100 m) the volume fraction of sand that could be transported was 0.145 without the high polymer additive and 0.25 with additive. Note that Figure 1 in Reference 4 presents 15 percent by volume as practical limit. The explanation lies in the pressure head available when a centrifugal pump is used as

the head is increased, the flow decreases. With a pneuma pump the air pressure can be increased within limits but the centrifugal pump requires a speed change–or a pump change.

(2) The head of slurry transport centrifugal pump is relatively low but the capacity is quite high. A 1976 survey (Audi and Pitts, 1976) points out that in hydraulic dredging, a head of 350 with a flow rate of 40,000 gpm through an 88-inch pipe is available.

(3) Particles that are large enough and robust enough to damage the pump impellers pass through the pneuma pump. Non-Newtonian slurries can be difficult to move with a centrifugal pump even though they could flow through the pipes. Large stringy particles can foul both pumps. (A sheet of plastic 6 feet wide and 20 feet long fouled a pneuma pump, but a suit of overalls did not.)

(4) Turbidity development in the waterway is not a function of the pumping technique specifically; rather, it is a function of the solids excavation made together with the pumping technique. If the cutter head stirs up a cloud of dispersed particles – turbidity – then the pump suction can only minimize particle dispersion. The cutter head must be designed with turbidity control in mind. Reference 5 has some interesting and pertinent comments about turbidity control.

In the two pneuma pump examples described above, a grid was pushed or dragged through the sediment piles. This method of engaging the solids does not develop turbidity in the waterway. If the "right" amount of water also gets through a high solids slurry results. A significant part of this paper is devoted to the description of a method for determining the "right" amount of water. Dragging of a grid through a sediment pile applies only to soft layer or layers of sediment. If the sediment pile is not soft, then other methods of turbidity control must be designed for either the pneuma pump or centrifugal pump.

(5) Wear of components is significantly different in the two pumping systems. Wear of the discharge piping is to a first approximation – a function of the total solids transported. Similarly, the wear of the inlet, or suction, pipes is a function of the total solids transported. The wear of the impellers in the centrifugal pumps can be relatively fast. In the pneuma pump the wear of the inlet and discharge check valves is quite low. The wear rate of the three-way air plug valve can be rapid. The valve design has been modified. On the whole, the pneuma pump can be low in the wear rate of its components.

(6) The energy usage required by a pneuma pump is higher than that required by centrifugal pump for the same slurry pumping task. Basically, the energy contained in the compressed air, at the end of the discharge cycle, is vented to atmosphere. R&D must be done to develop a method of conserving that energy.

(7) System control is not used extensively in dredging operations. Control is the most important concept that needs to be developed for efficient dredging. A mass flowmeter is needed whether efficient operation is defined as 15 percent solids ratio or 40 percent solids ratio. Probably it is even more important for centrifugal pump systems; since, if a drop of 10 percent occurs, then a low of five percent solids is being transported. A drop from 15 percent to five percent means that pumping takes three times as long for the same weight of solids. A drop from 40 percent to 20 percent means that twice as long is required. In the future, feedback control will be used with a mass flowmeter to maintain the desired solids ratio during slurry dredging.

(8) The total cost is determined by capital cost of the equipment, the time and material costs of installation and removal, the time and material costs of dredging together with the costs of environmental controls.

Table 3. Elapsed Time and Water Volume Required to Dredge 1000 Yards of Solid (C – percent solids by volume)

	C = 5%			C = 30%		
q_{sl} ft³/g	Δt hrs	q_w ft³/hr	V ft³	Δt hrs	q_w ft³/hr	V ft³
2	75	6,840	513,000	12.5	5,040	63,000
10	15	34,202	513,000	2.5	25,020	63,000
50	3	171,000	513,000	0.5	126,000	63,000

Table 2. Comparison of Dredging Parameters

			C** = 5%		C = 10%		C = 20%		C = 30%		C = 40%		C = 50%	
q_{sl}	D_{15} *	D_{10}	q_{sol}	q_w	q_{sol}	q_w	q_{sol}	q_w	q_{sol}	q_w	q_{sol}	q_w	q_{sol}	q_w
ft^3/s	in.	in.	yd/hr	ft^3/hr	yd/hr	ft^3/hr	yd/hr	ft^3/hr	yd/hr	ft^3/hr	yd/hr	ft^3/hr	yd/hr	ft^3/h
2	4.94	7.82	13.3	6,840	26.7	6,480	53.3	5,760	80	5,040	106.7	4,320	133	3,600
10	11.06	17.5	66.7	34,202	133.3	32,400	266.7	28,808	400	25,200	533	21,600	667	18,000
50	24.72	39.08	333.3	171,000	666.6	162,000	1,333	144,000	2,000	126,000	2,667	108,000	3,333	90,000

* The diameter of the slurry discharge pipe required for a mean slurry velocity of $\bar{u}$ = 6 & $\bar{u}$ = 15 ft/s.
**C is % solids by volume.

Table 4. Dredging Parameters

I. Slurry Flow Rate $q = 2.0$ ft³/g (900 gpm)

A. Slurry Lift h = 20 ft. Discharge length, L = 1000 ft

	$\bar{u}$ = 6 ft/s				$\bar{u}$ = 15 ft/s			
ρ	1.08	1.25	1.33	1.665	1.08	1.25	1.33	1.665
C, %	5	15	$\boxed{20}$	$\boxed{40}$	5	15	$\boxed{20}$	$\boxed{40}$
D, in	7.8	7.8	7.8	7.8	5	5	5	5
Sscfm[1]	200	205	207	217	400	431	445	504
P, hp[1]	10	11	11	13	55	65	69	88
Δp, psi	15	16	17	19	66	73	78	95
Δp, ft.	35	38	39	44	149	169	179	219
P, hp[2]	8	8.6	9	10	34	38	41	50

B. h = 100 ft. L = 2000 ft

	$\bar{u}$ = 6 ft/s				$\bar{u}$ = 15 ft/s			
ρ	1.08	1.25	1.33	1.665	1.08	1.25	1.33	1.665
C, %	5	15	$\boxed{20}$	$\boxed{40}$	5	15	$\boxed{20}$	$\boxed{40}$
D, in	7.8	7.8	7.8	7.8	5	5	5	5
Sscfm[1]	372	379	383	398	689	739	762	857
P, hp[1]	48	50	51	55	156	177	187	230
Δp, psi	57	59	60	64	155	173	181	215
Δp, ft.	131	136	139	148	$\boxed{357}$	$\boxed{399}$	$\boxed{418}$	$\boxed{496}$
P, hp[2]	30	31	31	34	81	90	95	113

C. h = 250 ft. L = 2000 ft.

	$\bar{u}$ = 6 ft/s				$\bar{u}$ = 15 ft/s			
ρ	1.08	1.25	1.33	1.665	1.08	1.25	1.33	1.665
C, %	5	15	$\boxed{20}$	$\boxed{40}$	5	15	$\boxed{20}$	$\boxed{40}$
D, in	7.8	7.8	7.8	7.8	5	5	5	5
Sscfm[1]	590	596	599	612	868	914	936	1023
P, hp[1]	118	120	121	126	235	257	268	312
Δp, psi	122	124	125	129	220	238	246	281
Δp, ft.	281	286	289	298	$\boxed{508}$	$\boxed{549}$	$\boxed{568}$	$\boxed{648}$
P, hp[2]	64	65	65	68	115	124	129	147

[1] Compressor intake & power

[2] Centrifugal pump power

Table 4, continued

II. Slurry Flow rate g = 10 ft /s (4500 gpm)

A. h = 20 ft L = 1000 ft

	$\bar{u}$ = 5.6 ft/s				$\bar{u}$ = 15 ft/s			
ρ	1.08	1.25	1.33	1.665	1.08	1.25	1.33	1.665
C, %	5	15	$\boxed{20}$	$\boxed{40}$	5	15	$\boxed{20}$	$\boxed{40}$
D, in	18	18	18	18	11	11	11	11
Sscfm[1]	889	897	901	918	1343	1416	1450	1587
P, hp[1]	34	35	36	38	115	130	138	170
Δp, psi	11	15	15	12	31	34	36	43
Δp, ft.	25	26	26	28	71	79	83	99
P, hp[2]	28	29	30	31	80	90	94	112

B. h = 100 ft L = 1000 ft

	$\bar{u}$ = 5.6 ft/s				$\bar{u}$ = 15 ft/s			
ρ	1.08	1.25	1.33	1.665	1.08	1.25	1.33	1.665
C, %	5	15	$\boxed{20}$	$\boxed{40}$	5	15	$\boxed{20}$	$\boxed{40}$
D, in	18	18	18	18	11	11	11	11
Sscfm[1]	1642	1648	1651	1664	2014	2076	2105	2225
P, hp[1]	183	184	185	188	282	300	308	344
Δp, psi	45	46	46	47	65	69	71	77
Δp, ft.	105	106	106	108	151	159	163	177
P, hp[2]	119	120	120	122	171	180	184	202

C. h = 250 ft L = 2000 ft

	$\bar{u}$ = 5.6 ft/s				$\bar{u}$ = 15 ft/s			
ρ	1.08	1.25	1.33	1.665	1.08	1.25	1.33	1.665
C, %	5	15	$\boxed{20}$	$\boxed{40}$	5	5	$\boxed{20}$	$\boxed{40}$
D, in	18	18	18	18	11	11	11	11
Sscfm[1]	2804	2814	2819	2840	3392	3461	3536	3725
P, hp[1]	537	541	542	550	761	789	820	900
Δp, psi	113	114	114	115	152	156	162	175
Δp, ft.	260	262	262	265	$\boxed{350}$	$\boxed{361}$	$\boxed{373}$	$\boxed{404}$
P, hp[2]	295	297	297	301	396	409	423	458

Pumps Solids Supply

A major problem in the overall design of a dredging system is that of feeding the pump – either a pneuma pump or a centrifugal pump. Contrast this design problem with the design of a backhoe – the ubiquitous tool in surface excavation. Backhoes are designed to have the capability to embed its teeth into the solid surface; it also is designed to have the power to scoop up the solid. If the excavator is not filling the backhoe bucket, the condition is obvious and the economic loss of waste motion is apparent. Since the dredge cutter – under water – is not visible, the lack of capability to maintain a high solids ratio is not apparent until the slurry is discharged hundreds of feet away.

If the interface between the leading element of the dredge and solids is favorable then a high solids ratio slurry can be pumped. If that interface is not favorable, then a low solids ratio slurry will be pumped (see Ref. 5). If the solid flows easily the initial slurry will have a higher percent solids ratio. However, if the leading element of the dredge is not moved, the solids ratio will continue to decline since the solid must travel further and further to reach the dredge. Water pumping is the default condition if some way of continually filling the pump is not designed into the system. If the leading element is moved stepwise, the slurry solids ratio will be spasmodic – high when the leading element is inserted into a new solids position with a steady decline until the next step move.

Technical Discussion
Pneumatic Pump Applications

Figure 4 presents a conceptual view of the methodology which will be discussed. (This particular concept is addressed to the Namtec pump. The methodology will be applied to other dredge pumping systems later in this paper.*)

In Figure 4, the discharge end labeled "To Pump" would connect to an inlet manifold which would be joined to the pipes labeled "Slurry Inlet" in Figure 1. When a slurry segment (A, B and C) is in either the discharge mode or the incipient discharge mode, the air pressure above the slurry (Figure 1a) will maintain the inlet check valve in a closed position. Only the segment C (Figure 1a) will accept slurry from the feeder (Figure 2).

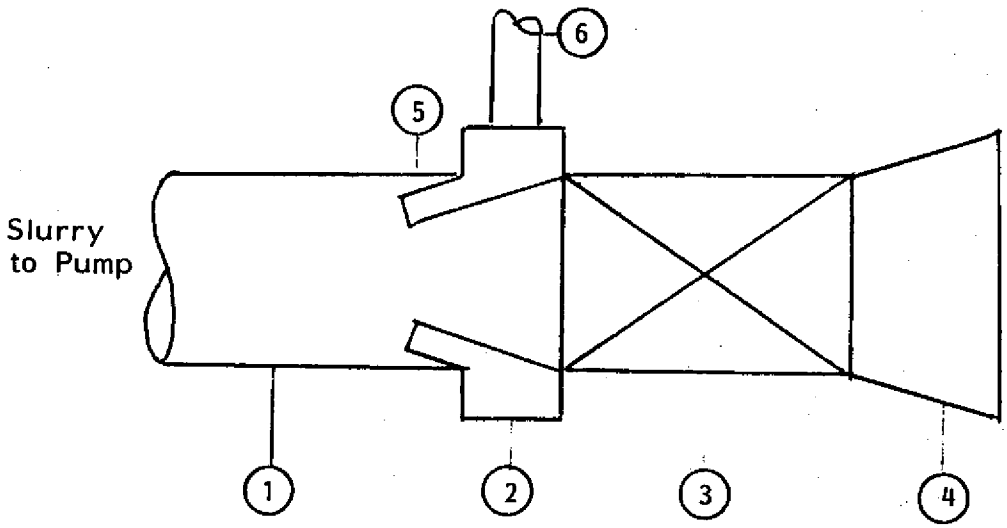
The manifold (Figure 4) serves to convert the waterhead pressure into relatively high velocity water jets. These jets both mix the incoming solids, from the screw, into a slurry and transport the slurry into the Pneuma pump segment. The slurry outlet from the manifold can either be rigidly piped to the pneuma pump or it could be designed as a flexible hose so that there could be relative motion between pump and the scoop, screw and manifold combination.

The dredging scoop (1) should be far enough below the water-solids interface so that water does not bypass from the interface, since that would entail pumping of water reducing the solids ratio – an economically detrimental side effect. The screw (2) meters the solids into the manifold (3) at the desired rate; and also provides a seal to prevent backflow of the slurry when a soft spot or cavity is present in the solids bed.

The manifold (3) provides for two functions: first, it receives the solids from the screw and second, it provides the place where the slurry is mixed by the jets of water coming from the manifold. The fluid dynamics will be discussed later. From the mixing zone the slurry enters the Pneuma feed pipe. Depending on the particular dredging requirements, this feed pipe could be either short and rigidly attached to the Namtec pump or of significant length and flexibility before its attachment to the Namtec pump.

The characteristic of the medium that is to be dredged, quite obviously, is of first order importance when the design of the cutter or other initial contact element of the design is considered. If a soft ooze or sludge is to be dredged a cutter could be dispensed with; however, if a hard sand bed is to be dredged, a cutter is necessary. Dredging occurs at a depth H below water level. There are no restrictions on H. The depth h below the benthic layer has a minor restriction.

At dredging depths of less than 12 feet, considerations of pumping assist must always be made. An important fact is that even if pumping assist is required, the pump would not be expensive. Since the pump would handle water only, an abrasion resistant pump would not be needed. The dredging depth required to fill the Pneuma pump chambers is only part of the technical problem of slurry pumping with a Pneuma pump. One must also consider the time requirement to fill the pump chamber. The chamber must fill rapidly so that the slurry discharge does not become sporadic. Thus the filling time and the discharge times must be essentially equal.



1 - Pipe to Slurry Pump
2 - Slurry Water Supply Manifold
3 - Solids Feeding Unit
4 - Solids Scoop or Cutter
5 - Slurry Mixing Jet Pipe
6 - Water Supply Pipe (from pump for Shallow Dredging)

Figure 4. Slurry Feeding Concept Schematic

Prevention of blowback through the digging element must be considered in the discussion of this methodology. When the Pneuma slurry pump is used with no water pump assist, (i.e., the dredging depth is 12 feet or greater), the head of water effectively prevents back flow from the jet slurry mixing manifold.

The screw is used to feed solids not to prevent back flow. However, voids in the dredging zone will be encountered; and then the screw will effectively prevent blow-back.

Conventional Dredging Applications

This methodology can be applied to conventional dredging where a centrifugal water pump is used to transport the solids to the settling area. However, the slurry transport pressure would be on the order of five to 20 atmospheres. A simple screw feeder would not be appropriate. Back-flow through the cutter would be continual. However, a rotary lock could be used. The major advantages would be the use of water pumps to pump water only.

The slurry injection concept described above can be applied to standard dredging operations with a few modifications. A screw feed could be used to supply the solid to the manifold, and the water jet would be used to lift the solid either to a barge or to the shore if the shoreline was close to the dredging operation. The main consideration is that the total lift would have to be acceptable to the sealing capability of the screw.

Another method to incorporate the slurry injector is to use a rotary lock. The rotary lock is used to inject solids into an air stream for pneumatic transport. The rotary lock is, of course, an added expense, but that expense is partially compensated for by the use of a less expensive pump. Further, a pump would be above water, reducing maintenance costs.

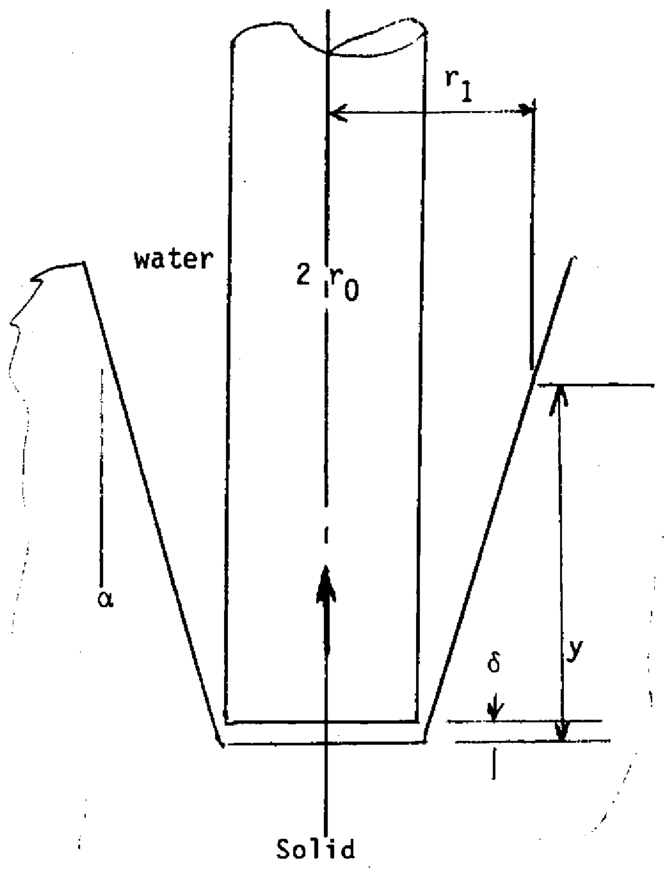
The existing system requires that the centrifugal pump act as a suction pump to entrain the solids and to pump the slurry through the pump. This design requirement imposes the necessity of abrasion resistance and the capability to transport large particles of solids. Furthermore, wear of the rotary lock – due to abrasion – would be much less than wear of the pump impellers because the rotary speed of the lock is much lower than the rotational speed of a pump.

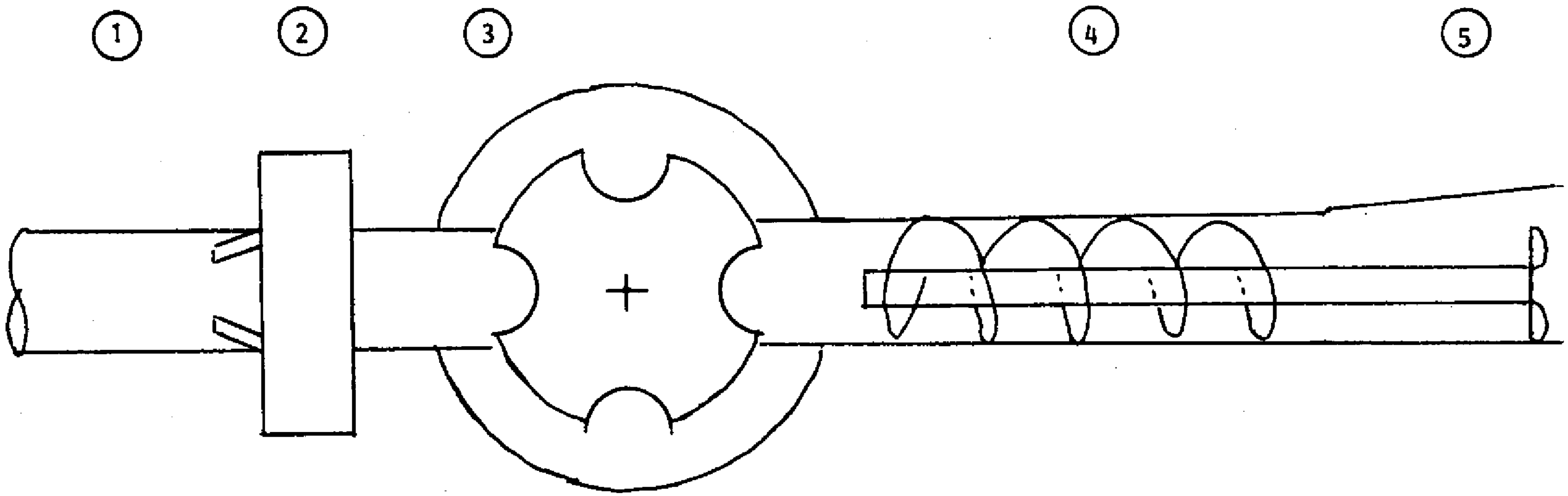
Figure 5 is a conceptual representation of a solids feeding lock. Consider Table 1 where the effects of 2, 10 and 50 ft3/s slurry transport are presented. We can extend that analysis by considering the effect on excavation rate for the different solids concentration. For the purpose of clarification approximate pipe sizes are included on the figure, i.e., for a slurry rate of qs1=2 ft3/s, the slurry pipe is 4 inches in diameter and the lockscrew feeder pipe is 12 inches in diameter. The advance rate of the scoop, for a 25-percent slurry flowing at 2.0 ft3/s is .5 ft/s or 0.4 mph. However, if the slurry flow rate is still 2.0 ft3/s but the slurry is only five percent solids then the excavation advance rate is reduced to 0.1 f/s or 0.08 mph. This discussion of the solids feeding lock applies to either the conventional system or a pneuma system.

Returning to the specific feeding requirements of the pneuma pump the problems associated with both cases of a free-flowing solid and a cohesive solid will be presented. The accompanying sketch shows the general case, where a slurry is formed from the solid on the bottom and the water above it. If the cohesive strength of the solid approaches zero the angle a would approach 99°; however, if the cohesive (or shear) strength of the solid is higher, then the angle a would approach zero degrees. For the case where a = 90°, the solid would be required to flow like a liquid; and in the case where a = 0°, the excavation would develop a bore hole slightly larger than the excavation pipe.

Free-Flowing Solid. There would be no need of a feeding device. One would just place an excavation tube in the solid and commence pumping slurry. The tube would have to be immersed in the tube so that x was the correct value to allow a slurry of the desired percent solids to enter. It is the author's opinion that such free-flowing solids are few and far between. The water head – or pressure – required to fill the Pneuma pump is the sum of the final height of the slurry in the pump chambers (h1); the velocity head required for slurry mixing (h2); and the friction of the slurry flow through the feed pipe (h3) between the manifold and the Pneuma pump will be less than 7 feet; the slurry mixing velocity head (h2) is dependent on the degree of mixing required. The friction head (h3) is dependent on the length and diameter of the slurry feed pipe.

Non-Free Flowing Solid. If the solid has some shear strength so that the angle a is between 90° and 0° then that cohesion must be broken to that a slurry can develop. In some cases a water jet can be used to erode the solids and permit them to enter the pump (either pneuma or centrifugal). In other cases, the shear strength of the solid is great enough that the solid is only eroded in the vicinity of the water jet and the water jet is not effective in the development of slurry for pumping.





1 - Transport Pipe to Slurry Pump
2 - Slurry Mixer Water Manifold
3 - Solids Feed Lock
4 - Solids Feed Screw
5 - Excavation Cutter

Figure 5. Solids Feeder and Lock

For the cases where the solid is not free-flowing and where the water jet is ineffective – perhaps the large majority of the cases – a mechanical excavator will be required. Depending on the cohesive strength, the excavator design will range from solids guidance to a rugged digger. Figure 4 shows an excavator followed by a rotary lock which is followed in turn by the jets required for slurry mixing. The mean flow rate, (u) of the slurry must be adequate to maintain particle suspension. The water velocity (uj) through the jet must be great enough so that the mean transport pipe velocity u is high enough to maintain particle suspension. Thus,

$$n\, A_j\, u_j = Au$$

where
n = the number of jets
Aj = jet area
A = transport pipe area
and $u_j = Au/(nA_j)$

However, uj must be high enough to supply the energy required to break up clumps of solids as well as mixing the solids into a slurry.

A pressure head is required to fill the pump – whether it is a pneuma or a conventional centrifugal pump. This pressure head can be due to either the suction developed by the centrifugal pump or the lowered pressure to venting the pneuma pump segment. The pressure head required to cause slurry to flow through the discharge pipe is

$$\Delta p = \lambda \frac{\rho}{D} \frac{\bar{u}}{2}$$

where
x = pressure head friction factor
L = length of pipe
x = slurry density
$\bar{u}$ = mean slurry velocity
D = pipe diameter

This equation is applicable to either the inlet or the discharge pipe.

L is the equivalent length of the pipe, including fittings, valves, flowmeters, etc. D is the local inner diameter of the pipe of length L. u is the mean slurry at the local diameter, D.

The minimum u depends on the type of material, the size and shape of the particles and the uniformity of particle sizes. The value of the friction factor is well defined for the flow of fluids as

$\underline{N_R}$ laminar flow, $= N_R^{-1/4} \times 0.316$

$N_R = \frac{\rho \bar{u}}{\mu}$ turbulent flow

It is usually assumed that the slurry is a continuous liquid in that its viscosity is definable separately from the viscosity of the carrier fluid (usually water). Actually the solid particles do not have viscosity; the carrying water does. The water impinges on the particles and "drags" around them. If the flow is turbulent then vortices develop at the boundary (the pipe surface) and help keep the particles in the flow stream.

The definition of x is the trickiest part of the analysis. The flow will be either Newtonian or non-Newtonian. Some non-Newtonian flows are not easily handled by centrifugal pumps. Due to the different mode of pressure application, a pneuma pump can handle flow whether it is Newtonian or not.

If the type of fluid is new to the dredger he should determine the slurry viscosity preferably at the mean velocity (u) and pipe diameter that is to be used in his dredge.

Conclusions

The slurry pumping capabilities and limits of the pneuma pump have been presented. The pneuma pump has been evaluated against the conventional centrifugal pump. The centrifugal pump is less expensive and does not require as much energy for the same slurry pumping job. However, the centrifugal pump cannot pump slurries with a high percent solids. A single pump cannot deliver as high a head. For small and mid-sized dredging jobs, the pneuma pump can be an efficacious choice.

Acknowledgment

I want to acknowledge to assistance given to me during the preparation of this paper. Mr. Austin of Mudcat was helpful in locating this source in which to present my paper and was encouraging during preparation. Mr. D. Harris of Namtec supplied the photos that were used and helpfully reviewed the first draft. Finally, I am glad to take this opportunity to thank Ms. Nancy Robinson of Robinson's Office Services for typing and polishing this paper.

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Biodata

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Evaluation of the Matchbox Dredgehead and Submerged Diffuser

T.N. McLellan, C.L. Truitt and M.R. Palermo
Waterways Experiment Station

Introduction

Resuspension of sediments and possible release of contaminants is a concern when dredging highly contaminated sediments. The Waterways Experiment Station (WES) has initiated studies to determine the relative effectiveness of methods of dredging contaminated sediments under the Improvement of Operations and Maintenance Techniques research program. WES has also recently participated in a study of alternatives for dredging and disposal of sediments from two highly contaminated reaches of the Indiana Harbor, Indiana channel for the Chicago District (Environmental Laboratory, 1986). Indiana Harbor is located on Lake Michigan, near Chicago, Illinois. As a part of the Indiana Harbor evaluations, a demonstration of innovative equipment was conducted to provide data for possible application to the Indiana Harbor project. The demonstration was a cooperative effort of the Corps' Water Resources Support Center, North Central Division, Chicago District and WES. Innovative equipment used in the demonstration included a Dutch-designed matchbox hydraulic dredgehead designed to minimize resuspension of sediment during operation and a submerged diffuser for controlled placement of material, both conceptually illustrated in Figure 1. The performance of the matchbox dredgehead was compared with performance of a conventional hydraulic cutterhead and clamshell dredge.

Matchbox Dredgehead

Special-purpose dredging systems have been developed during the last few years in the United States and overseas to pump dredged material slurry with a high solids content and/or to minimize the resuspension of sediments. Most of these systems are not intended for use on typical maintenance operations; however, they may provide alternative methods for dredging project having highly contaminated sediments such as in Indiana Harbor. The major drawbacks of special-purpose dredges are their limited availability and their inability to be incorporated into conventional transport and disposal operations. The Dutch matchbox dredge (Figure 2) can, however, be incorporated into an operation similar to a cutterhead suction dredge.

The matchbox suction head is designed to dredge fine-grained material as close to in-situ density as possible, keep resuspension to a minimum while dredging layers of varying thickness, and operate with restricted maneuverability (d'Angremond, de Jong and de Waard, 1984). To keep resuspension to a minimum, all cutter and waterjet devices commonly found on dredgeheads were avoided.

Several innovative design features are incorporated into the matchbox dredgehead construction. These design features include:

(a) A plate covering the top of the dredgehead to contain escaping gas bubbles and avoid the influx of water.

(b) An adjustable angle constructed between the dredgehead and ladder to maintain the optimum dredging position regardless of dredging depth.

(c) Openings and valves installed on both sides of the suction head so that the leeward opening can be closed to avoid water and sediment release.

(d) Dimensions of the dredging plant which are carefully designed to account for the average flow rate and swing speed of the dredge.

The matchbox dredge may be suitable for dredging the contaminated sediments located in Indiana Harbor. Not only can the matchbox head be incorporated into a conventional cutterhead dredge operation, but this device has been shown to produce suspended solids concentrations of less than 135 mg/l (d'Angremond, de Jong and de Waard, 1984). The matchbox head accomplishes this while dredging the sediments close to in-situ density.

Submerged Diffuser

Placement of contaminated sediments in an open-water site and subsequent capping with cleaner sediment is a possible disposal option in many cases. The amount of water column turbidity generated by an open-water pipeline disposal operation or barge pumpout can probably be minimized most effectively by using

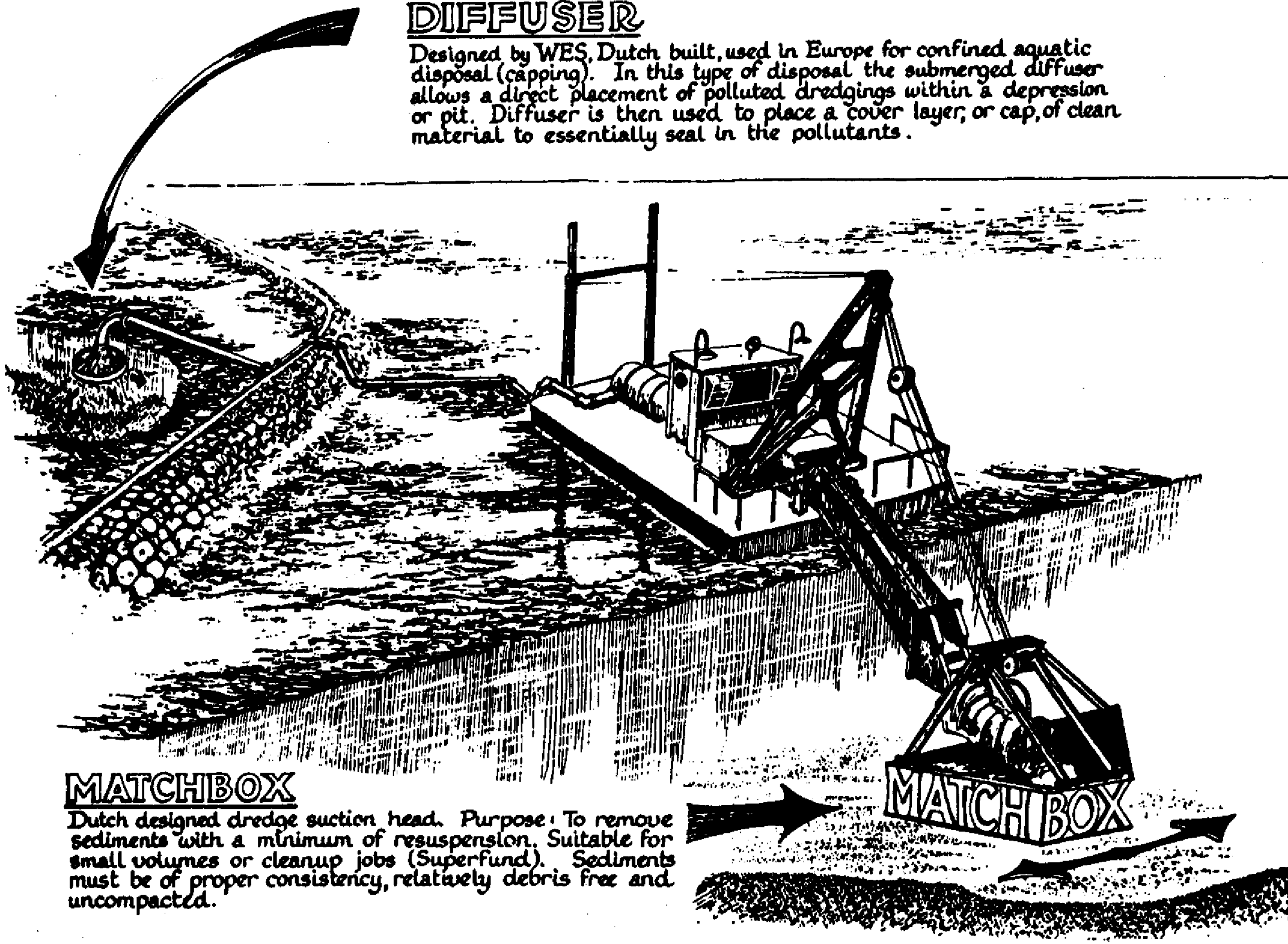

Figure 1. Conceptual illustration of innovative equipment demonstration at Calumet Harbor, Illinois (Provided by U.S. Army Engineer District, Chicago)

Figure 2. Photograph of matchbox dredgehead

a submerged diffuser system (Figure 3) that has been developed through extensive laboratory flume tests conducted under the Dredged Material Research Program (Neal, Henry and Greene, 1978). This system has been designed to eliminate all interaction between the slurry and upper water column by radially discharging the slurry parallel to and just above the bottom at a low velocity. The entire discharge system is composed of a submerged diffuser and an anchored support barge attached to the end of the discharge pipeline that positions the diffuser relative to the bottom.

The primary purpose of the diffuser is to reduce the velocity and turbulence associated with the discharged slurry. In one design, this is accomplished by routing the flow through a vertically oriented, 15-degree conical diffuser with a cross-sectional area ratio of 4:1 followed by a combined turning and radial diffuser section that increases the overall area ratio to 16:1 compared to the pipeline. Therefore, the flow velocity of the slurry prior to discharge is reduced by a factor of 16, yet the dredge's discharge rate (i.e., slurry flow velocity X the pipeline cross-sectional area) is not affected in any way by the diffuser. The conical and turning/radial diffuser sections are joined to form the diffuser assembly, which is flange mounted to the discharge pipeline. An abrasion-resistant impingement plate is supported from the diffuser assembly by 4 to 6 struts. The parallel conical surface of the radial diffuser and impingement plate slope downward at an angle of 10 degrees from the horizontal so that stones and debris can roll down the sloped surface and automatically clear the diffuser. The radial discharge area of the diffuser can be adjusted by changing the length of the struts supporting the impingement plant. In this manner both the thickness and velocity of the discharged slurry can be controlled. The strut length, which determines not only the slurry discharge velocity but also the maximum diameter of an object that will pass through the diffuser, should be approximately five-sixths of the pipe diameter.

A discharge barge (Figure 4) must be used in conjunction with the diffuser to provide both support and the capability for lowering the diffuser. The barge also provides a platform for the diffuser while it is being adjusted, serviced or moved to a new site.

The diffuser has a great deal of potential for eliminating turbidity in the water column and maximizing the mounding tendency of the discharged dredged material. The slurry remains in the pipeline/diffuser until it is discharged at low velocity near the bottom, or below a zone of high current velocity, thus eliminating all interaction of the slurry with the water column above the diffuser.

Equipment Demonstration

Demonstrations of a clamshell dredge, a cutterhead suction dredge, the Dutch matchbox dredge and a

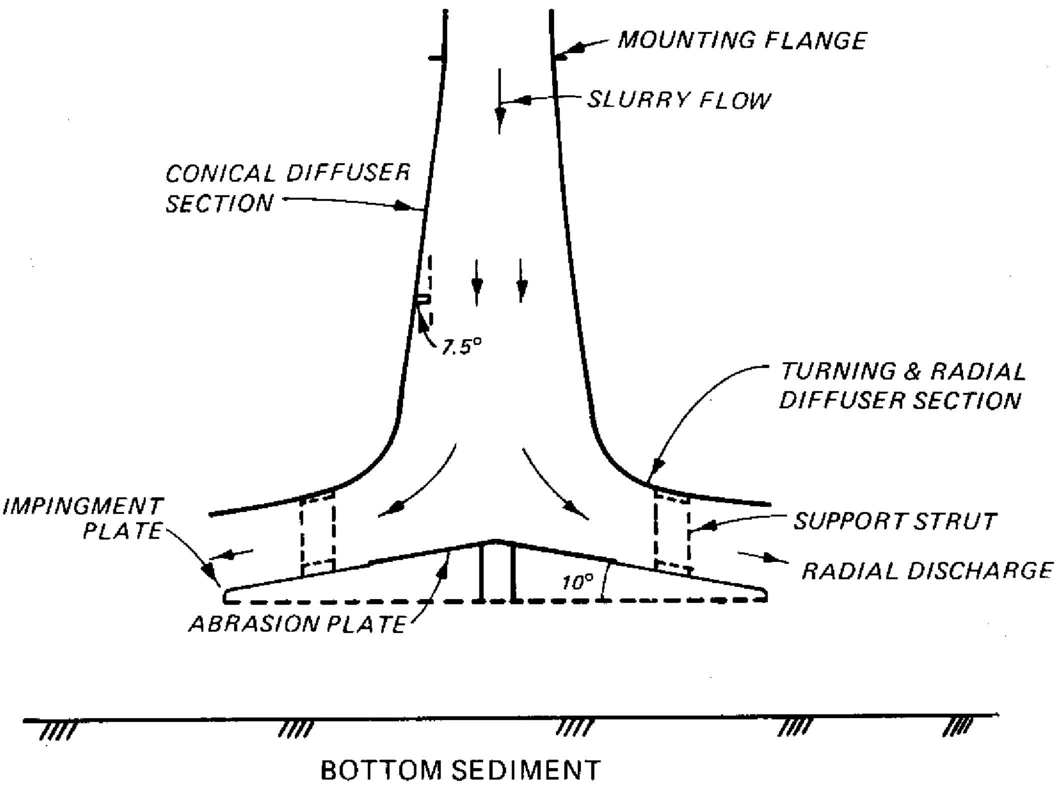

Figure 3. Submerged diffuser system

submerged diffuser were conducted in the Chicago District in August through October of 1985. The demonstrations were conducted in Calumet Harbor, which is just north of Indiana Harbor on Lake Michigan. Water depths at Calumet were approximately 35 to 40 feet. Sediment samples and current measurements collected at Calumet Harbor and Indiana Harbor indicate that the physical parameters at each site are similar. Therefore, the results obtained from the Calumet Harbor equipment demonstration can be directly applied to Indiana Harbor. The equipment demonstrations included field monitoring efforts developed under the IOMT Program to measure suspended solids, dredge production and possible release of contaminants. The detailed results of these equipment demonstrations will be submitted in a separate report to the Chicago District (Hayes, McLellan and Truitt, 1986).

Clamshell field evaluation. The clamshell dredge demonstration was conducted during maintenance dredging occurring in Calumet River. The monitoring effort included water sampling to define the size and concentration of the suspended solids plume, observations of the dredge operating characteristics, collection of water samples for chemical water quality analyses and sediment sampling to be used for elutriate testing and bulk sediment analysis. The clamshell dredge field study incorporated one day of background sampling and two days of plume monitoring in the interior Calumet River. A total of 13 sampling stations at varying distances from the dredging operation were used and samples were collected at near bottom, mid-depth and near surface. The field study identified a suspended sediment plume with a suspended sediment concentration at least 10 mg/l above ambient of 3.5 acres near the bottom, 1.8 acres at mid-depth, and 1.7 acres near the surface (see Table 1). This 10 mg/l level also corresponded to approximately twice the concentration of the ambient suspended sediment concentration. The rapid reduction in area of the plume from bottom to mid-depth indicates that the plume is generated primarily by the impact, penetration and withdrawal of the bucket from the sediment. The highest concentrations and greatest variability of the plume were found near the bottom where samples collected within 50 feet of the dredge ranged from 540 mg/l to 49 mg/l.

Hydraulic dredge field evaluation. The 12-inch hydraulic dredge DUBUQUE was used to compare performance of the matchbox dredgehead and a conventional cutterhead. The cutterhead demonstration was

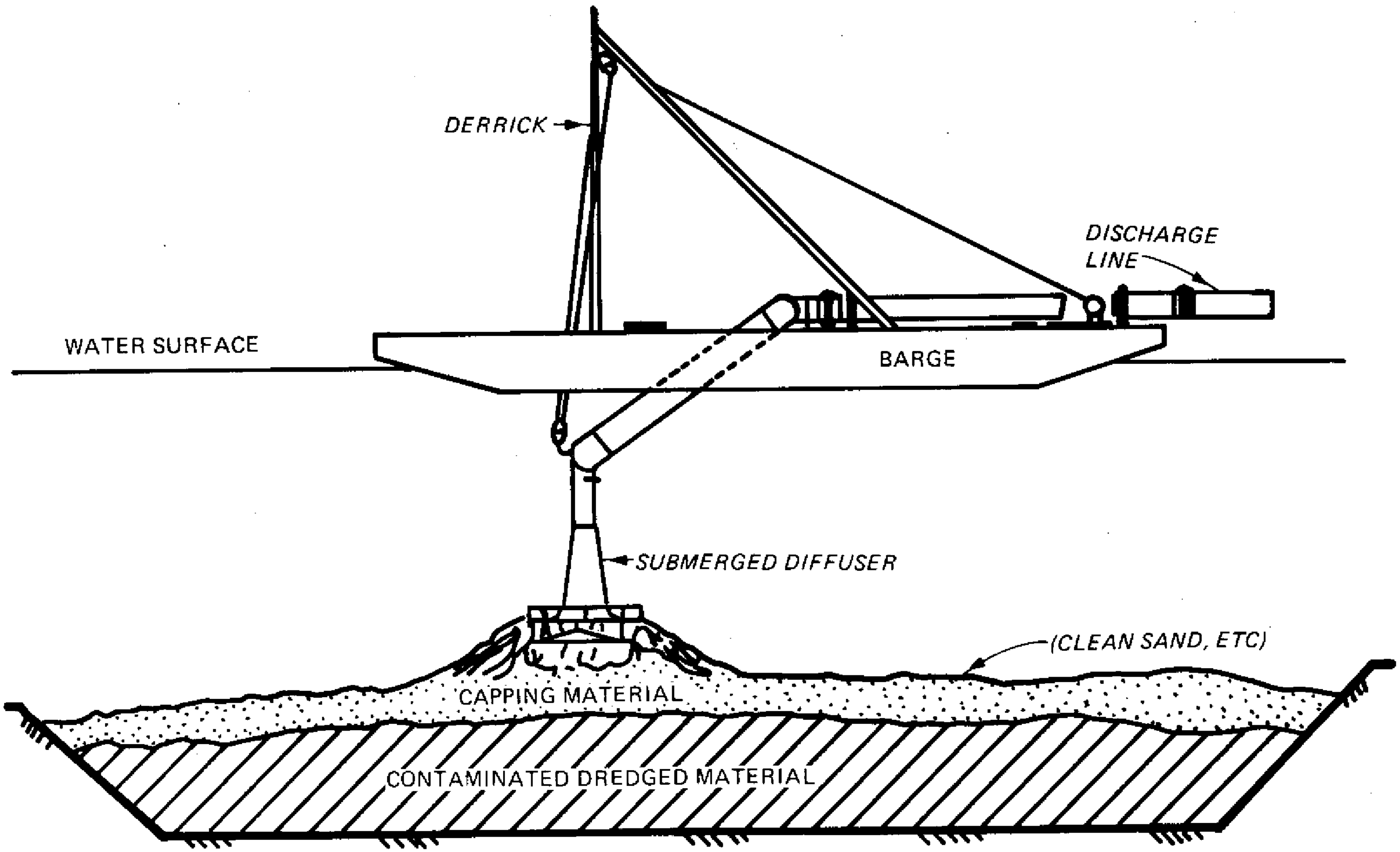

Figure 4. Submerged diffuser system, including the diffuser and discharge barge.

conducted in Calumet Harbor near the Chicago Area confined disposal facility (CDF). The monitoring plan included observations of the cutterhead operation and collection of discrete water samples to measure suspended solids. The cutterhead operational parameters measured included production rate, swing speed, cutter rotational speed and depth of each cut. The discrete water samples were collected from a specially designed head sampler attached to the dredge's ladder, which allowed collection of samples within 5 feet of the cutterhead. Additional water samples were collected at 6 to 10 stations located in and around the dredging operation at 5, 50, 80 and 95 percent of the total water depth. The field demonstration of the Dutch matchbox dredge was also conducted at Calumet Harbor. The dredge was the same one used in the cutterhead suction demonstration, except that the cutterhead was removed and the matchbox head installed. The monitoring plan was similar to that used for the cutterhead dredge. The dredge head sampler was modified since the matchbox has no cutter, but the operation of both dredges was similar. The demonstration of the matchbox suction head dredge was the first use of the dredge in this country.

Two days of background sampling preceded the two days of matchbox testing which was followed by another day of background sampling and three days of cutterhead testing. A suspended sediment plume with a concentration of at least 10 mg/l above ambient was identified for the matchbox operation over an area of 2.9 acres at 90 percent of the total depth and 0.4 acres at 80 percent of the total depth. No plume of this concentration was identified above this depth. Similarly, a suspended sediment plume with a concentration at least 10 mg/l above ambient of 1.2 acres was identified for the cutterhead operation at 95 percent of the total depth (see Table 1). No plume of this concentration was identified above this depth for the cutterhead operation. All sampled concentrations of suspended sediment in both plumes at distances of 100 feet or greater were all less than 20 mg/l except for a few observations.

Table 1. Cross Sectional Area of Suspended Solids Plumes with Solids Concentration Exceeding 10 mg/l Above Ambient

	Near Surface	Mid-depth	Near Bottom
Clamshell	1.7	1.8	3.5 (90% depth)
Cutterhead	—	—	1.2 (95% depth)
Matchbox	—	—	0.4 (80% depth) 2.9 (95% depth)

Submerged diffuser field evaluation. The submerged diffuser demonstration was conducted simultaneously with the matchbox dredge demonstration (Figure 5). The demonstration site was inside the Calumet CDF located near the mouth of the Calumet River. Water depth in the CDF at the diffuser location was approximately 20 feet. The submerged diffuser demonstration was designed to evaluate the effectiveness of the diffuser in reducing the velocity of the dredged material and limiting the suspended solids plume to the lower portion of the water column. Velocity measurements were obtained at the exit of the diffuser and at a station located 7.5 feet from the diffuser exit. During the demonstration, pipeline velocities were reduced 75 to 80 percent at the diffuser exit and the diffuser's ability in containing the suspended solids plume to the lower portion of the water column was displayed. At a station 12.5 feet from the diffuser exit in 20 feet of water, water column samples were collected at increments of 5, 50, 80 and 95 percent total depth, every 5 minutes throughout the dredging period. With ambient TSS concentrations averaging 4 mg/l, the average TSS level for the 5 and 50 percent depth samples was 9.6 mg/l, while the average of the 80 and 95 percent depth samples in the discharge path was 3,266 mg/l. The diffuser was able to significantly reduce the slurry velocity, confine the discharged material to the lower portion of the water column, and reduce suspended sediment effects in the upper portion of the water.

Discussion and Potential Application

Based on the results of these demonstrations, both cutterhead and matchbox resulted in far less resuspension than the clamshell dredge (see Table 1). The tests showed that the cutterhead can remove sediment with very little resuspension when operated properly. The data for the cutterhead operation shows very low levels of resuspension near the cutterhead. Additional analysis of the cutterhead data may provide insight to the impact of operational parameters on the resuspension process.

The matchbox dredgehead performed very well from the standpoint of production considering the operator's inexperience in using the dredgehead. The matchbox is also capable of removing sediment with very little resuspension. However, the data for the matchbox operation reflected precise positioning problems. The operator could not determine when the top of the matchbox was at the same level as the sediment nor could he properly match swing speed with flowrate. These are important for optimum operation of the matchbox. The data which did not appear to be so affected shows very low levels of resuspension near the matchbox. Consequently, before the matchbox suction head could be recommended over the cutterhead for removing contaminated sediments, additional studies need to be conducted with better control of the matchbox position relative to the bottom. Improved instrumentation for density and flow measurement and computer controls linked directly to the dredge engines for orientation of the matchbox are available (Taylor, 1986). Such equipment could significantly improve the efficiency and effectiveness of the matchbox.

The submerged diffuser was able to reduce the pipeline exit velocity by 75 to 80 percent. However, the exit velocities were three to four times greater than the theoretical predictions. Additional investigations may be needed to evaluate these variations. The demonstration clearly showed the diffuser's ability to limit sediment resuspension to the lower portion of the water column. The diffuser was able to significantly reduce the slurry velocity, confine the discharged material to the lower 20 to 30 percent of the water column, and reduce suspended sediment effects in the upper portion of the water.

The dredging alternatives chosen for a particular project depend on, but are not limited to, availability of equipment, disposal site selected, dredged material contaminant levels, hydraulic characteristics of the area, and physical characteristics of sediment. Using the DMRP and IOMT research programs as background and the results of the demonstrations, several innovative dredging alternatives have been identified for the Indiana Harbor Project. The dredging alternatives include use of an enclosed clamshell bucket, a cutterhead dredge operated under specific guidelines, and a Dutch matchbox suction head dredge. Transport techniques to reduce sediment resuspension include proper care when handling, replacing and extending pipelines for hydraulic dredge operations and special loading and disposal techniques for barge transport. Disposal techniques that could be incorporated into the Indiana Harbor Project include the use of the dredged material and reduce suspended sediment levels associated with disposal operations.

Acknowledgment

The tests described and resulting data presented herein, unless otherwise noted, were obtained in part from research conducted under the Improvement of Operations and Maintenance Techniques research program of the United States Army Corps of Engineers by the Waterways Experiment Station. Additional data were obtained from studies conducted for the Chicago District of the Corps of Engineers. Permission was granted by the Chief of Engineers to publish this information.

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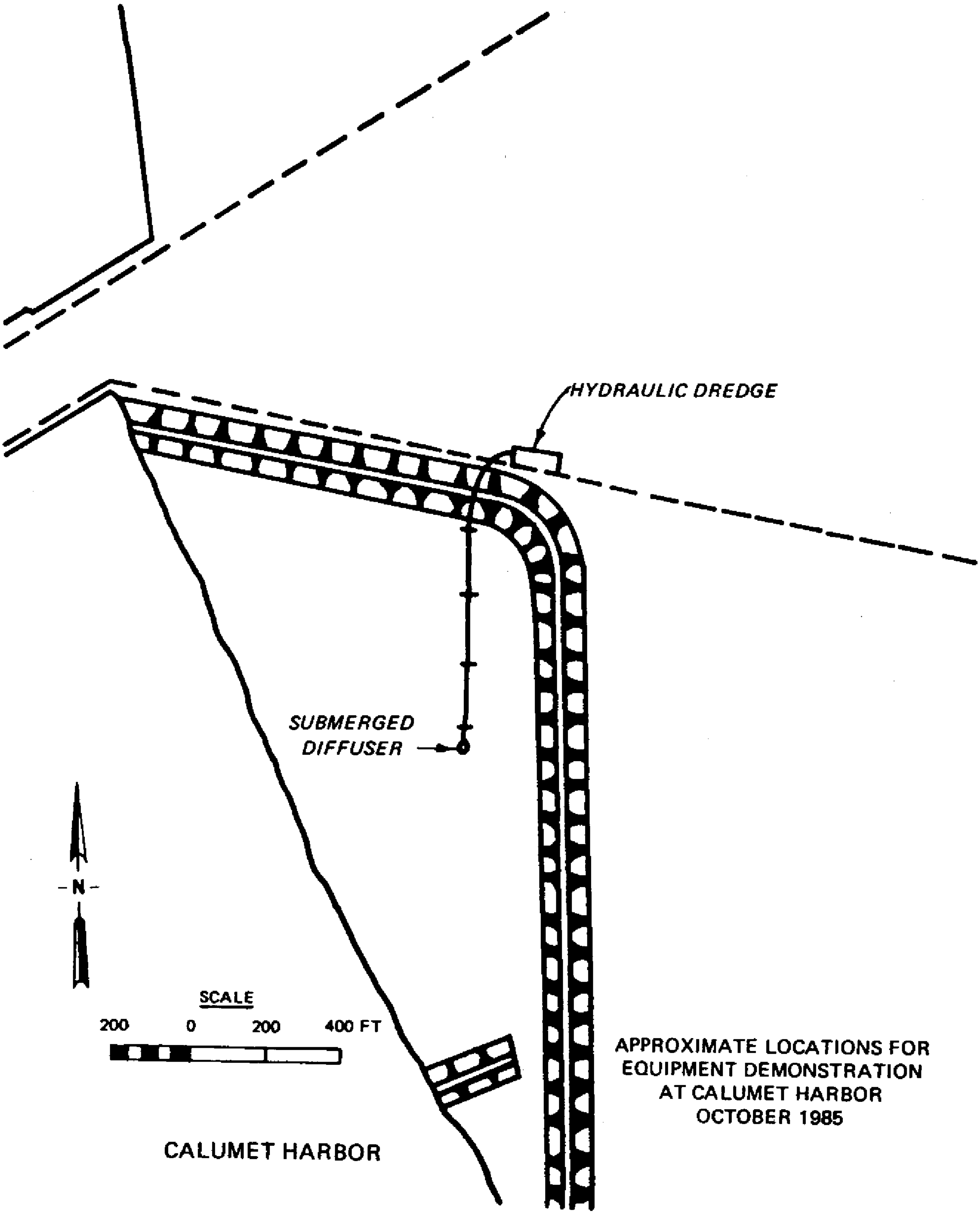

Figure 5. Submerged diffuser field demonstration location

Biodata

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During his tenure at the Waterways Experiment Station, Dr. Palermo has been a principal participant in the Corps of Engineers' Dredged Material Research Program, Environmental and Water Quality Operation Studies, and other major research programs within the Environmental Laboratory. He remains actively involved in studies concerning various aspects of dredging and dredged material disposal technology, including design, operation and management of containment areas, fine-grained dredged material dewatering, and marshland development using dredged material. He is also currently involved with research concerning dredging and disposal of contaminated sediments. Prior to his position at WES, Dr. Palermo was a civil engineer with the Vicksburg District, Corps of Engineers, beginning his career with the Corps in 1967.

Dr. Palermo is a member of the Vicksburg Engineers Club, Chi Epsilon, Society of American Military Engineers and American Society of Civil Engineers. He is a registered professional engineer in the state of Mississippi.

The Effectiveness of a Twenty-Inch Dredge in Thin Layer Disposal

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U.S. Army Corps of Engineers
Mobile District

Introduction

Fowl River is a small coastal stream in the western shore of Mobile Bay in south Mobile County, Alabama. In 1973, the U.S. Army Corps of Engineers constructed an 8- by 100-foot channel for commercial fishing and recreational boating interests. At the time of construction, open water and wetland areas (where diked disposal areas were constructed) were used for dredged material disposal. The project has been maintained three times in total and once in partial, each time experiencing disposal area problems, to wit, a lack of adequate dredged material disposal storage capacity. These problems were highlighted by Hurricane ELENA in 1985 where destruction of a "just completed maintenance job" at the mouth of the river caused renewed emphasis to establish a long-term maintenance disposal plan for this project. The Mobile District, in coordination with federal and state regulatory agencies, devised a plan whereby a combination of upland and open water disposal methods would be utilized during maintenance in 1986. The open water methodology involved what is called "thin-layer" disposal and, in this case, the thin lift after disposal was to be no greater than approximately six inches. This disposal process required dispersal of the Bay Channel material over a large open water surface. The dispersal is defined "theoretically" by making the assumption that every grain of dredged material removed from the channel and pumped to the disposal area would drop directly out of suspension to the disposal area bottom of the particular deposit point. It was felt that this thin-layer deposition would have less short- and long-term impacts to the Mobile Bay aquatic ecosystem than conventional open water disposal and if it could be done cost effectively would provide an acceptable long-term dredging and disposal plan for many other small navigation projects in the Mobile District. As part of this project, a year-long monitoring has been initiated to determine the physical and biological impacts of the thin-layer disposal, the results of which will be published upon completion. This paper will discuss the actual process of achieving a thin-layer.

Equipment

The portable dredge GEORGIA, otherwise known as the PD-20S, was used to dredge Fowl River in July-August 1986. The GEORGIA was built in 1980 by the American Marine and Machinery Company, Inc. (AMMCO) for the state of Georgia for the maintenance dredging of the state docking facilities at Brunswick and Savannah. In July 1986, River Products, Inc. of Norco, Louisiana, secured the dredge for use in the Fowl River job. The GEORGIA is shown in Figure 1 and described in Table 1.

Dredge pipeline used during the project was a combination of 20-inch plastic pipe and steel pipe. The plastic pipeline utilized for this job was manufactured by J.W. Christie, Inc. of Frisco, Texas, and each pipe was 20 feet long, 22 inches outer diameter, and 19-7/8 inches inner diameter.

Plastic dredge pipeline connection was achieved by the use of a J.W. Christie, Inc., "Butt Fusion Joiner" which is a 230-volt, 9,800-watt, 45-amp heating device that heats to temperatures of 500°F (Figure 2). Dredge pipeline was assembled on shore in 600-foot lengths and then towed to the required area for connection to the dredge. The plastic pipe was connected to two steel, ball and bell, vertical swivels mounted on pontoon (mud bug) barges which were 30 feet long by 20 feet wide (Figure 3). The first swivel was located approximately 600 feet from the dredge and the second swivel was located 3,800 feet from the first swivel or 4,400 feet from the dredge. Both swivels were anchored in position.

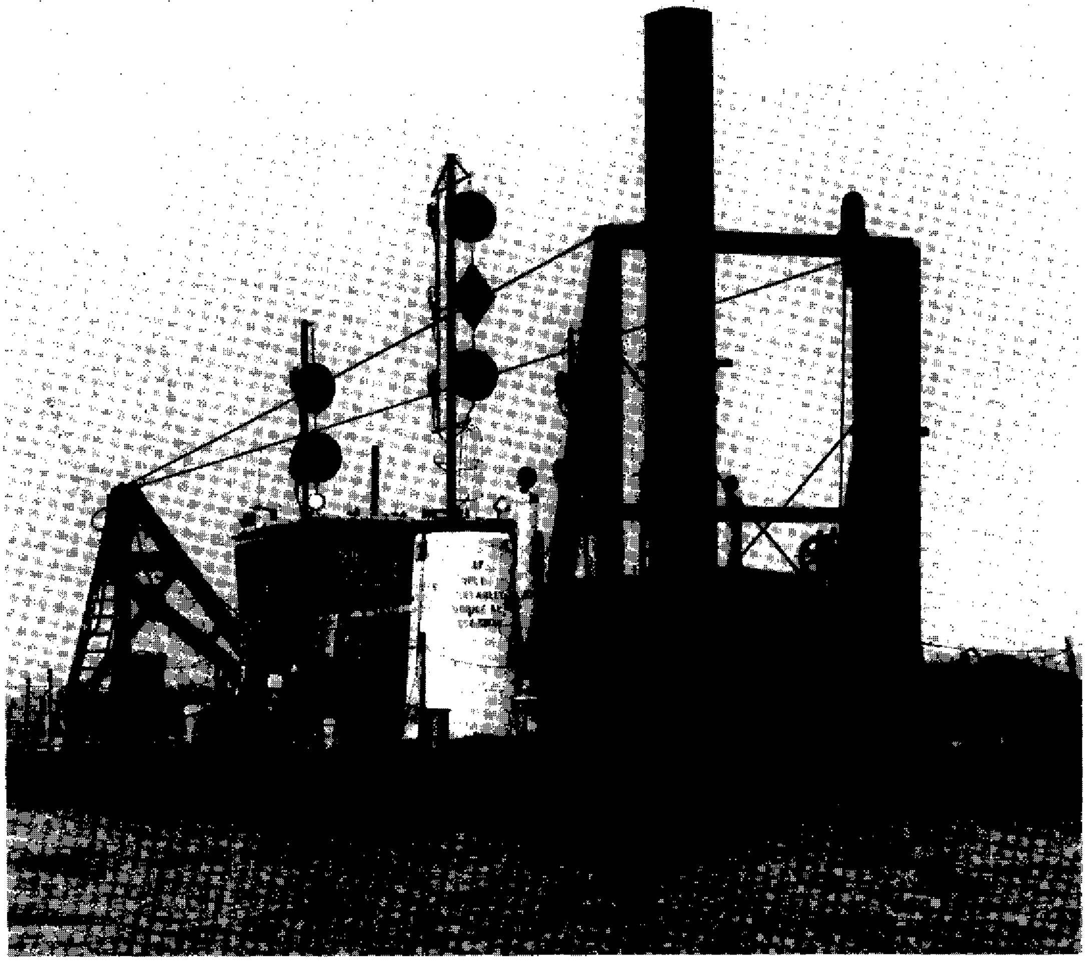


Figure 1. The portable dredge GEORGIA

Table 1. Description of the Dredge GEORGIA

DREDGE MODEL/SERIES: PD-20S

MANUFACTURER: American Marine and Machinery Company, Inc. (AMMCO)

GENERAL:	
Length	70 feet
Width	22 feet
Weight	213,606 lbs.
Draft	41 inches
Fuel Capacity	8,000 gal.
PUMP:	
Type	Centrifugal
Main Pump Horsepower	1,125 hp
Capacity	Not available
Suction Diameter	22 inches
Discharge Diameter	20 inches
CUTTER ASSEMBLY:	
Type	Cutterhead, 6-blade basket, Florida Foundry
Horsepower to Cutter	250 hp
WORKING CAPACITY:	
Digging Depth	To 54 feet
Production Rates	400-900 cu yd/hr (this job, 25 day average was 496 cu yd/hr)
Pumping Distances	To 4,000 feet
ANCHORING SYSTEM:	
Type	Spuds and winches

TRANSPORT/ASSEMBLY EQUIPMENT NEEDED:

Information not readily available; however, dredge transported to this job via barge.

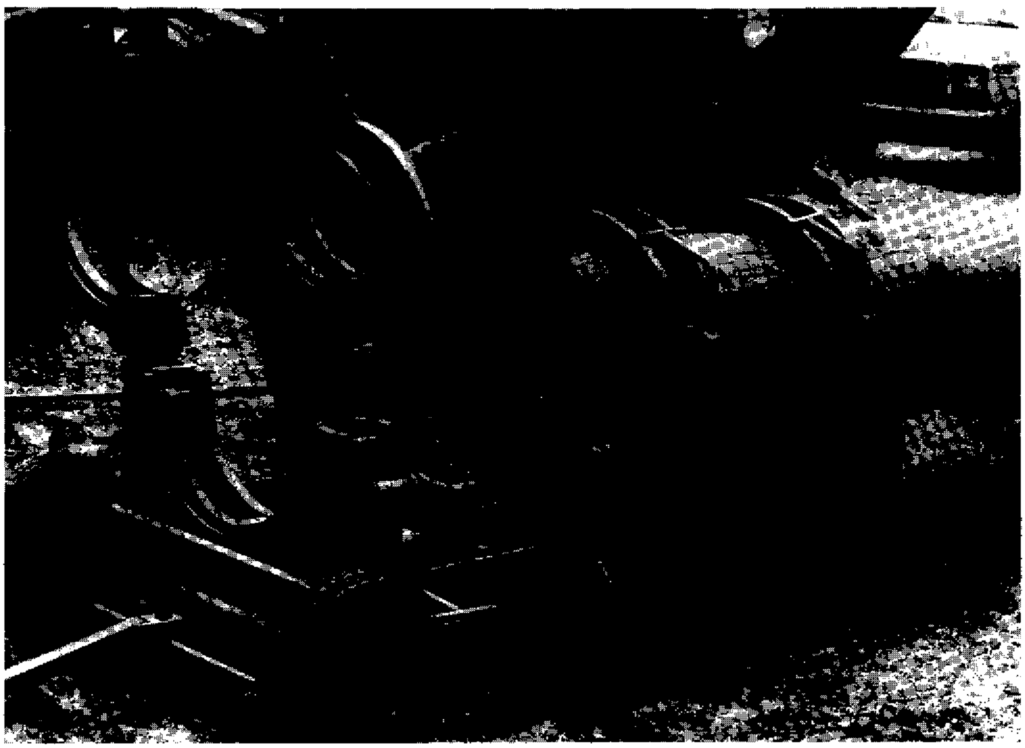


Figure 2. The J.W. Christie, Inc., "Butt Fusion Joiner"

The discharge pipe was mounted on a floating barge approximately 200 feet from the second swivel via a ball joint (Figure 4). The discharge barge was 40 feet long, 15 feet wide with a draft of 30 inches. These modifications were necessary for barge mobility and spreading of dredged material. The barge and discharge modification is shown in Figure 5. The end of the discharge pipe was fitted with a spreading device as shown on Figure 6 and Figure 7 (in operation).

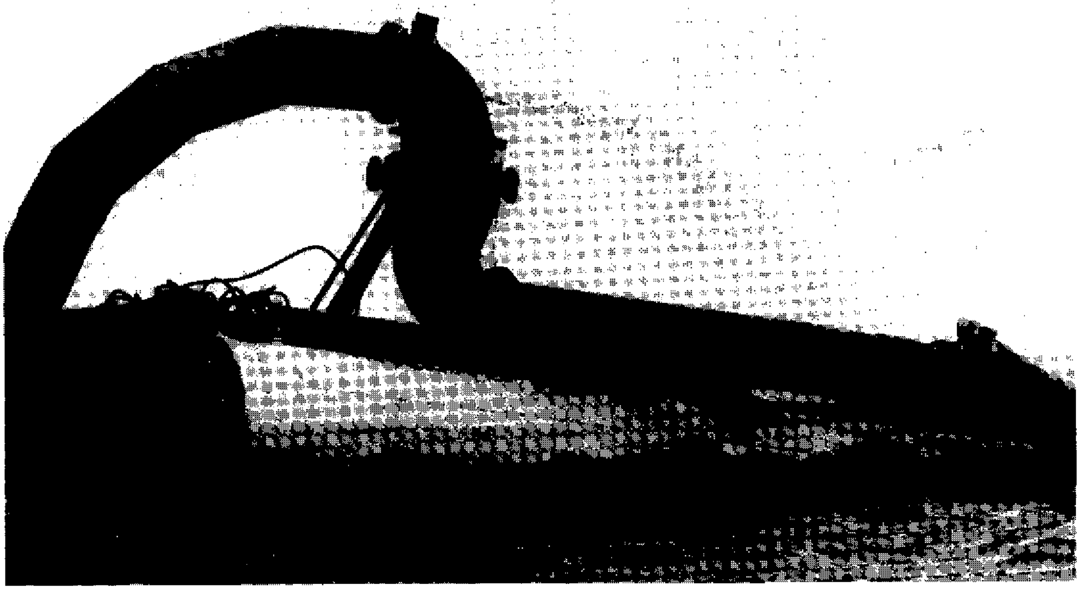


Figure 3. Vertical swivel on pontoons

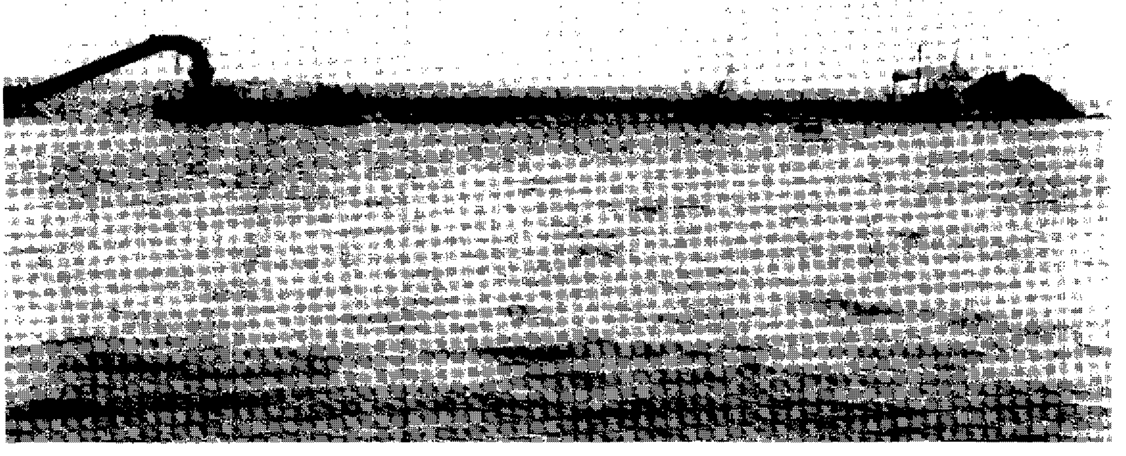


Figure 4. Vertical swivel, ball joint, discharge barge setup

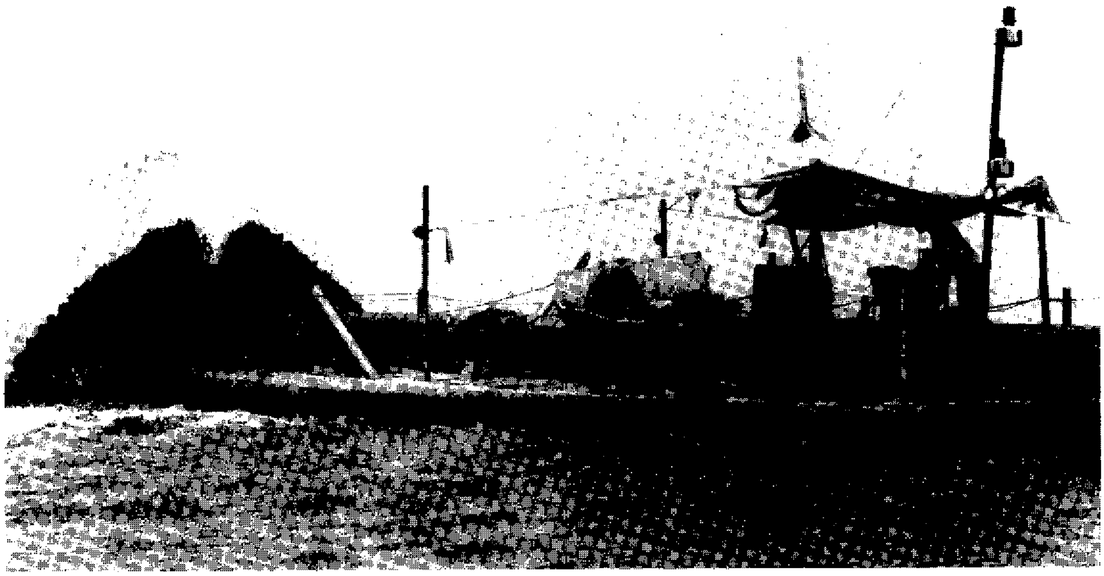


Figure 5. Discharge barge and spreading structure in operation

The spreading device consisted of a wing-mounted baffle plate located just in front of the pipeline slurry discharge flow. The baffle plate wing structure is of all steel construction and consists of two 3-foot sections of 6-inch H-beam welded to the top and bottom of the discharge pipe. The H-beam sections are fitted with a 4-foot by 3-foot section of steel plate (3/4-inch thick) which is pinned at the top and bottom.

The baffle plate wing is connected with steel cable to winches mounted on either side of the discharge barge which are operated by means of a small diesel engine equipped with a hydraulic pump. The spreading device discussed above is the final configuration of the discharge pipe and was the configuration found to be most productive during the disposal operation. Other configurations were attempted early on in the disposal operation, but these did not prove to be effective.

Figure 6. Discharge barge with view of spreading device components

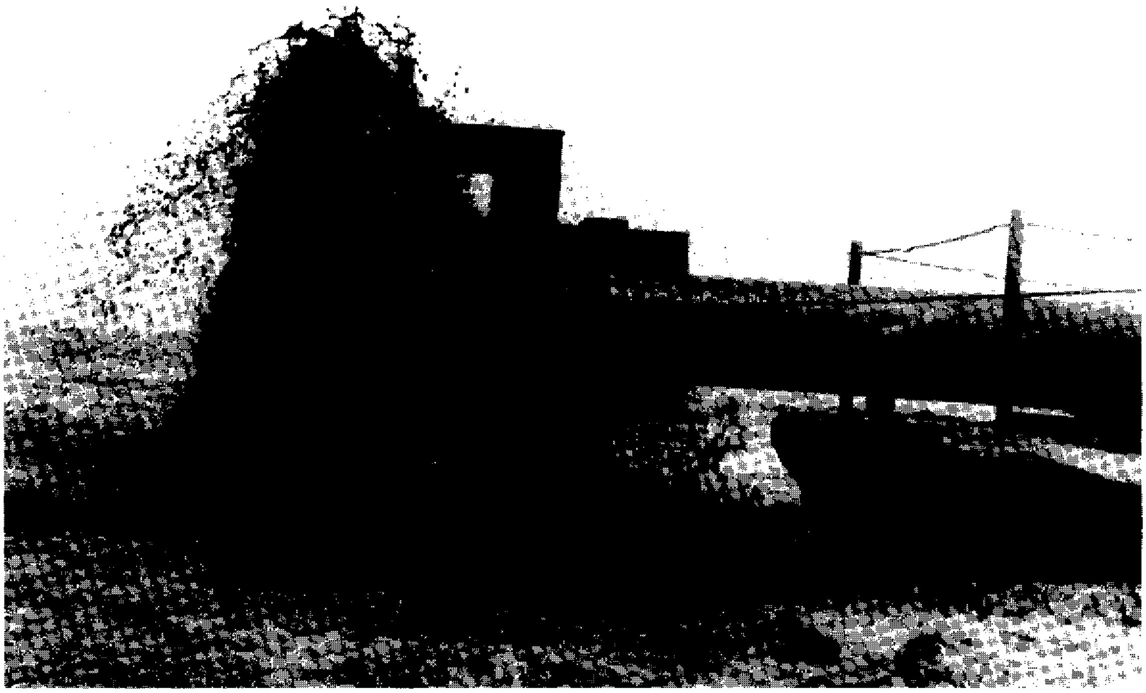


Figure 7. Spreading device in operation

Disposal Methodology/Operation

As previously stated in the Introduction, this report covers the actual process of achieving a "thin-layer" of dredged material, and, therefore, concerns only the open water disposal area.

The Fowl River Navigation Project was divided, for purposes of dredging, into two portions: (1) the outer bay portion of the project which incorporated about 7,000 feet of channel and (2) the river portion which incorporated about 7,300 feet of channel. The dredging requirement of the outer portion (the primary address of this paper) was for the removal of about 190,000 cubic yards of material, estimated to be 40 percent sand, 50 percent silt and possibly 10 percent sandy clay. Disposal of this material was in a previously used open water disposal area in Mobile Bay located just south of the channel. The area in which dredged material was placed is approximately 240 acres in size, but the actual disposal area (total area impacted by the discharge of material) was somewhat larger. The river portion involves about 90,000 cubic yards of material, primarily sandy silt, to be placed in a diked upland disposal area.

The open water disposal area was located in Mobile Bay, south of the Fowl River Navigation Channel (Figure 8). The northern limit of the disposal area was located 1,050 feet south of the channel and parallels the channel for approximately 4,000 feet. The southern limit of the disposal area was not defined. All material within the channel limits from Station 60+51 (western end) through 132+00 (eastern end), estimated to be 190,000 cubic yards, was to be deposited into this disposal area to a theoretical thickness not to exceed six inches. Approximately 240 surface acres of the disposal area were used in order to accomplish the theoretical six-inch layer.

The channel limits which contained the 190,000 cubic yards was 7,149 feet in length with a bottom width of 100 feet, and five-foot horizontal to one-foot vertical side slopes. The initial dredging, Station 96+92 to Station 68+92, was 2,800 feet in length and contained approximately 108,200 cubic yards of predominantly silty sand, with light silt and some clayey material, 38.6 cubic yards per foot of channel, which was required to be dredged to a depth of 14 feet below MLLW. The continuation of this cut to Station 60+51, the western limits of the bay channel, is 841 feet in length and contains approximately 40,000 cubic yards of sandy silt material, 47.6 cubic yards per foot, which was required to be dredged to a depth of 10 feet below MLLW. Based on these averages, it was assumed that the dredge would advance in the channel at the rate of 24 linear feet and 20 linear feet per operating hour, respectively, dredging approximately 945 cubic yards of material per hour. The dredge would then be relocated to Channel Station 132+00, the eastern limits, and would proceed westerly to tie-in to Station 96+92, the original starting point. This portion of the channel is 3,508 linear feet

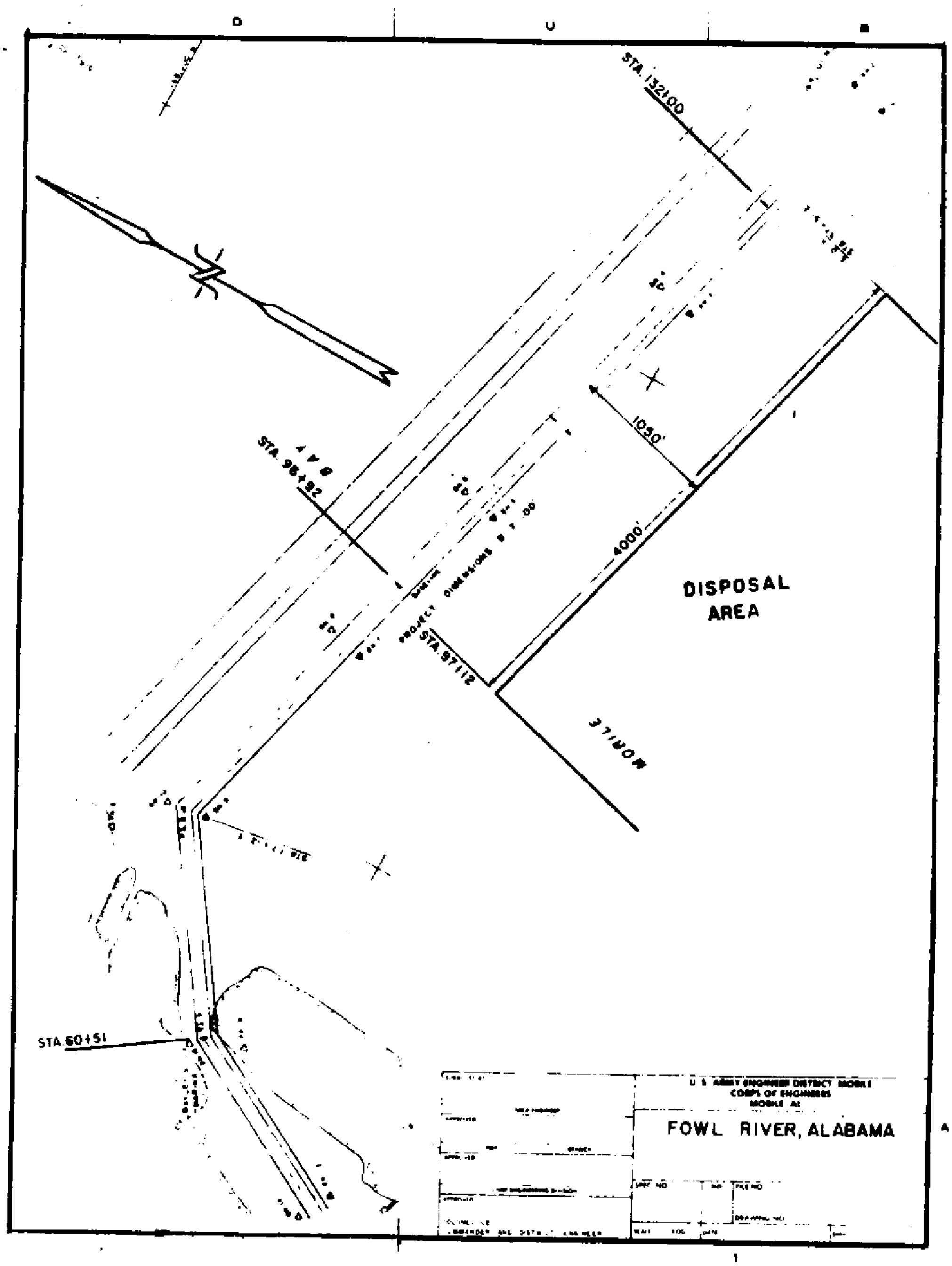


Figure 8. Bay Portion of Fowl River Navigation Project and Disposal Area

long and contains only 41,800 cubic yards of material. The dredge was expected to progress at a rate of 50 linear feet per operating hour in the channel, dredging approximately 600 cubic yards per hour.

The dredge discharge pipe was placed as shown in Figure 9 with the first point of discharge designated with the numeral "1." Station Number 1 was located approximately 1,500 feet south of channel Station 128+00. As indicated earlier, the end of the line was steel dredge pipe approximately 200 feet long with a baffle spreading device (Figures 6 and 7) on the outer end and a swivel joint at approximately 200 feet from the outer end (Figures 3 and 4). The discharge was moved by means of the baffle which was attached to winches on the discharge barge, and, when needed, with the assistance of a shallow draft tug. The discharge pipe was allowed to move by the swivel joint in an arc of about 300 degrees. From the initial location, the discharge barge was moved in increments of 400 feet to the southwest, as shown in Figure 9, toward the western limits of the disposal area in hopes of reaching a point approximately 3,000 feet south of channel Station 100+00. However, at channel Station 111+12, the plastic pipe separated and the dredge was required to temporarily shut down. The broken section of plastic pipe was removed and taken to shore for repair, and modifications in the discharge plan were made which required that the next point of discharge be moved to the northwest to about channel Section 109+00. The dredge continued to proceed toward channel Station 96+92 and discharged the material, as shown on Figure 8, in the northwest portion of the disposal area. Upon reaching channel Station 96+92, the repaired pipeline was replaced and the discharge barge was relocated 3,600 feet south of channel Station 137+00.

While the swivel joint is stationary at its various locations, the discharge barge moved in a 200-foot radius, 300 degree arc around the swivel joint. The swivel joint was relocated at generally one hour intervals while the dredge was operating in order to assure dispersal of the material. The large arc of the discharge line was reversed to the northeast toward the north limits of the disposal area and back to the west limits while the dredge progressed in the channel. Due to the rectangular configuration of the disposal area, the length of the arc was reduced in size as the discharge was moved to the northwest. Even though the arc was reduced in some areas, the discharge barge was moved at hourly intervals.

Actual performance of the work resulted in productions of 11 and 18 linear feet of channel advance per operating hour; and 487 and 497 cubic yards of material per hour in the 14-foot dredge depth and the 10-foot dredge depth channel sections, respectively. When the dredge completed the bay channel to its western limits, approximately 78 percent, or 184 acres, of the disposal area had been utilized. The dredge began working on July 26, 1986, and completed the job on August 27, 1986. The discharge barge was moved a total of 41 times during the job.

The discharge barge was manned, using one person per watch, supplemented by two or more persons to assist in the movement and locationing of the discharge barge. Constant radio communication existed between the discharge barge and the dredge.

Disposal Results/Discussion

A detailed bathymetric survey was made of the disposal area prior to the disposal of dredged material in June 1986 by Taxonomic Associates, Inc. of Mobile, Alabama, in conjunction with the monitoring of the environmental impacts of the disposal operation. The surveys of the disposal area in the post-disposal condition are scheduled to be initiated during the week of September 15, 1986, and, therefore, will not be presented in this paper. However, as part of the dredging contract and for purposes of this paper, a condition survey was made of the disposal area before and after its utilization. The position of the discharge barge was located by means of survey equipment four times daily. These data were plotted on the map of the disposal area as shown in Figure 9 and, in this way, no significant discharge overlaps were caused. Interim soundings and surveys were made as the disposal progressed to monitor the thickness of material. Adjustments to the disposal method were made based on the interim surveys.

The condition surveys made during the disposal operation were recorded by means of a Raytheon 719-B depth recorder which generally has a tolerance of about six inches. Since this sounding device has a tolerance of six inches, which is the theoretical thickness of material sought in the disposal area, its effectiveness may be subject to question. Also, this sounding device is quite sensitive to high levels of turbidity, a parameter which can be quite high during a disposal operation. Nevertheless, this device was utilized on regular occasions to obtain some preliminary data as to the thickness of material. It should be noted that a more sophisticated method of determining the sediment thickness will be employed by Taxonomic Associates, Inc. during their post-disposal monitoring operations.

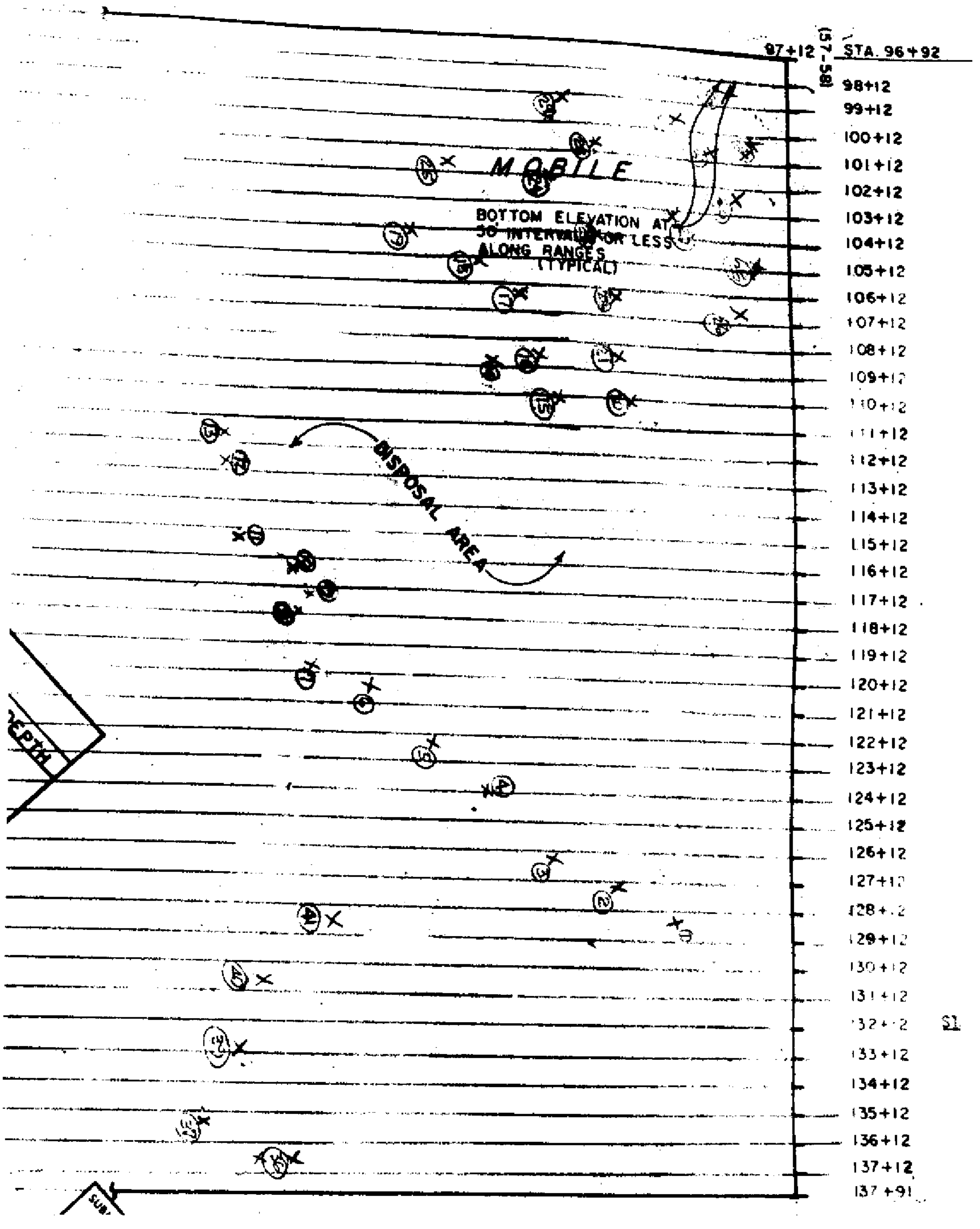


Figure 9. Project Worksheet with Discharge Points Plotted

The surveys conducted during the disposal operation with the use of the depth sounder were accomplished at a distance of about 150 feet from the point of discharge. Turbidities being fairly high, it was often difficult to determine sediment thickness; however, it appears that little or, in some areas, no buildup of disposed material above the six-inch level was accomplished. Those areas in which little buildup was noted were directly associated with the dredging of new work material in that section of channel which had been dredged to 14 feet below MLLW. Also, the discharge barge operator could clearly see the impact of the clayey material on the spreading device. While some increases above the six-inch level were noted, they appeared to be confined to small areas immediately around the point of discharge and not over large areas.

While all data pertaining to the effectiveness of the thin layer disposal operation are not available, it is evident from the preliminary surveys that a twenty-inch dredge can economically achieve a thin layer disposal with dredged material. Important factors to be considered in the application of this technique include: the type of material to be dredged, whether the material is new work material or maintenance material, and the type of spreading device to be used in the operation. Additional data on thin-layer disposal as it pertains to environmental impacts on the aquatic ecosystem will be presented at a later date.

Introduction to Cost-Sharing Panel

W. R. Murden
Water Resources Support Center
U.S. Army Corps of Engineers

Introduction

Good morning, I am Bill Murden, chief of the dredging division of the Water Resources Support Center of the U.S. Army Corps of Engineers.

It is a great pleasure to serve as the moderator for this Cost-Sharing Panel Session today.

The panel includes four speakers, representing different points of view, and I am sure we will enjoy hearing their presentations. I will introduce the first speaker shortly – but before I do so, let me take a few minutes to discuss cost sharing in a general sense.

In my view, cost sharing will become a reality. We hope that H.R. 6, the omnibus water resources bill, which includes quite a number of deep draft navigation projects, will be passed by the Congress and signed by the President in the immediate future. However, whether it is this year or next, I am convinced that cost sharing will come about and I believe it is a good idea for at least two reasons:

First, with the enormous federal deficit, the treasury simply cannot absorb the large expenses associated with the deepening of the channels serving our major coastal ports.

Second, the sharing of the deepening cost by the port authority and the associated state, as well as maintenance dredging associated with the deepening work, will contribute to a more stringent screening process. This, of course, will certainly contribute to improvement programs with a high level of success potential.

Cost sharing will result in a difference in how we, in the Corps, conduct our business in the navigation field. In the past, with 100 percent federal funding, we have had complete control over the design and also the construction processes. With the ports putting up 50 percent of the cost for deep draft projects, they will want, deserve and demand a say in the plans and designs. For example, it is already clear that phased construction based on a reasonable interval, say a 15-year forecast rather than a 50-year forecast, is in the cards.

Terminology – Let me address this topic for just a few moments.Terminology is very important to the dredging profession. Ninety-five percent of the material we dredge is not polluted or contaminated. Therefore, we do ourselves a disservice when we refer to dredged material as "spoil." We need to convey a positive impression whenever the facts will sustain a positive position. Let us stop using the word "spoil" in the dredging arena.

I recently attended a workshop on the beneficial uses of dredge material in Pensacola, Florida, which included many resource agencies and others having an interest in the environment. I am most pleased to announce that the vast majority of the presentations were very positive. It is important that we think and talk in positive terms when we refer to dredged material.

Well, these are just a few thoughts to get things underway.

Now it is my pleasure to introduce our first speaker of the panel – Mr. Richard Mors of the APAA – the American Association of Port Authorities. Mr. Eric Stromberg was scheduled to be on the panel but he and others in Washington, D.C., are doing water whatever can be done to urge the Congress to approve H.R. 6.

Biodata

William R. Murden, Jr.

William R. Murden, Jr. was born in Beaufort, North Carolina and is a graduate of Randolph Macon Academy. He attended The Citadel prior to serving as a command pilot during World War II. He received a degree in mechanical engineering at Elizabethtown College, Elizabethtown, Pennsylvania, and a master of business administration degree at Heed University, Hollywood, Florida.

He is chief of the dredging division of the U.S. Army Corps of Engineers Water Resources Support Center, a component of the Directorate of Civil Works. Mr. Murden has the staff responsibility for the dredging program of the Corps of Engineers, including the planning, budgeting, design and construction of Corps of Engineers dredges and other major floating and land plant operated in conjunction with the Civil Works program.

Mr. Murden is a member of the National Academy of Engineering. He is a registered professional engineer in the District of Columbia and Louisiana; chairman of the Corps of Engineers Marine Engineering Board; chairman of the Corps of Engineers Committee on Dredging Technology; honorary chairman of the board of directors of the World Organization of Dredging Associations; honorary chairman of the board of directors of the Western Dredging Association; and chairman of the finance committee of the Permanent International Association of Navigation Congresses. He is also a member of the American Society of Mechanical Engineers, the Society of American Military Engineers, and the Society of Naval Architects and Marine Engineers.

Cost Sharing and Its Impacts

Mark D. Sickles
Institute for Water Resources

The recently passed water resources development bill will have drmatic effects on the kinds of projects that the Corps of Engineers will build in the future, how they will be built, and where they will be built. Increased cost sharing is the primary agent of change but there are other provisions in the law that will ahve a significant impact on the Corps program. This discussion will cover some of these important sections as well as the final cost sharing percentages. Title VI (Senate version H.R. 6) potentially provides a radical departure from the status quo in the planning and implementation of harbor channel projects. The external recommendations that were accepted by the legislators represent the frustrations of the port development community of having to wait some twenty odd years to get a harbor improvement. Some (or most) of these frustrations may be alleviated with a new charter and the Corps' own administrative initiatives. (All sections of Title VI, Sections 213, 218, 223, 224 and others will be covered.)

The Initial and Expected Effects of Increased Cost Sharing

In last year's Supplemental Appropriation (FY 85), 13 harbor projects and four inland locks were either authorized, appropriated for, or both. This appropriations bill provided the first big test of administration policies. The Secretary of the Army was given broad discretion to reach agreements with project sponsors on the cost sharing terms. However, members of Congress understood that the now famous Senate Majority Leadership/Administration compromise on cost sharing would be the basis for reaching these agreements (called Local Cooperation Agreements, LCAs). Nine months were alloted to get the LCAs signed. The deadline proved to be a useful tool. Ten harbor channel project LCAs out of a possible eleven were "negotiated" successfully (two of the thirteen projects were not far enough along in the planning process for project commitments to be made). Six of these ten LCAs were for less than the total authorized project; in other words they were for first "phase" of the authorized project (Norfolk, New York, Baltimore, Mobile, Tampa, Mississippi River Ship Channel). These six harbors signed up for an average of 36.7 percent of the total estimated project cost. The agreed-to costs in these "phased-in" project LCAs range from $8 million to $300 million. (The authorized project costs ranged from $58 million to $486 million.) Building projects in phases makes good business sense for the ports because not only are the costs kept manageable, but the risk of overinvesting is lessened dramatically.

The frustration felt across the country each year about this time – when the federal government goes on the verge of shutting down for the lack of funds – had its positive side this year as the 99th Congress came to a very productive conclusion. The budget struggle turned out to be a stroke of good luck for those pushing stalled legislation. In Congress' closing days, a handful of important bills managed to squeak through despite (or because of) many legislators' preoccupation with the 1987 budget. The water resources development bill was one of these lucky bills. Had the scheduled October 3 adjournment not been delayed for 15 days by budget deliberations, months of work on H.R. 6 would probably have been lost (the bill passed overwhelmingly on October 17, 1986, as the last measure taken up by Congress and was signed into law as P.L. 99-662 on November 17, 1986, by the President.

Over the past few years the pressure to build several long-needed but unauthorized projects had led to a situation where project authorizations were somewhat regularly being made though appropriations bills. This type of legislative outcome has grown with the continuing budget morass, increasing the ability of the already powerful appropriations committee to set the nation's agenda. This short-circuiting of the traditional water development process created an uncomfortable situation for many. For the first time in history, the administration offered its own version of water omnibus legislation in an attempt to break the years-old deadlock over cost sharing and user fees. The unwelcome excursion into the traditional jurisdiction of the public works committees eventually helped lead to the grand compromise of June 1985. The Senate majority leadership sat down with then budget director David Stockman to hammer out the cost sharing percentages that eventually survived the legislative process. The new law is a victory for the authorizing, or law making, committees after years of struggling to resolve cost sharing issues.

The legislative history of the new law is noteworthy especially for the perception of it held by the interested publics and political alliances. Gone was the term "pork barrel" except as an unholy reference to past water omnibus bills. Editorial writers and politicians hailed the legislation as a major reform that will give new life and credibility to the water resources development program. The broad support, evident in the votes of both houses, was further demonstrated by the endorsement of the major environmental interest groups. The cost sharing provisions are viewed by them as a method to either eliminate the development of unwise projects or as an incentive to downsize the projects that are built. Furthermore, the trend toward the more haphazard and less visible action by both money committees, being considerably less friendly to environmental concerns, was another incentive to support H.R. 6. For the budget-conscious administration (and Congress), the conventional wisdom is that cost sharing is the ultimate test of efficiency. If those for whom the project is designed to benefit are willing to contribute a substantial portion, one has more confidence that the investment is justified. While this assumption makes common sense, and will certainly prove true in most cases, "real world" politicians at all levels are often willing to make risky decisions with taxpayers' money in the name of economic development. They may also be very willing to contribute to a project that does not have positive national economic development benefits (a benefit-cost ratio greater than one). This divergence of objectives could be the source of future disagreements between the Corps and its new non-federal partners.

The challenges ahead for the Corps of Engineers are many. Cost sharing rules are a blunt instrument that will spark challenges to many years of engineering practice and administrative procedure. Problem solving will probably be more difficult. In the past, many disputes could be settled with expensive solutions that may no longer be financially feasible; creativity will be in demand. Moreover, since project sponsors will naturally want to have greater impact on the resulting projects, a formidable challenge will be to productively bring them into the decision making process. Feasibility studies, now to be cost shared on a 50-50 basis, will provide the process for considering sponsor desires. However, creating a "new (planning) partnership" while at the same time adhering to the many other laws that have not been repealed, will ensure at least a degree of federal domination. The good news is that the goal of increasing the efficiency of the planning program by resolving the most critical issues in the earliest stages of "partnership planning" appears attainable. The bad news is that even if the new planning framework is successful in shortening the study period, there remains a problem with Congress. The authorization and appropriations process needs to be more dependable to gain the confidence of skeptical sponsors. Sponsors may be hesitant to commit scarce funds to a study only to have to wait ten years for Congress to act on the project. Congressional and administration staff are aware of the potential risk in damaging the government's credibility in this area. Whether meeting federal commitments becomes a problem remains to be seen. (H.R. 6 authorizes over $16 billion in projects, yet limits federal obligations to between $1.4 and $1.8 billion annually over the next four years.)

Another dilemma that faces the Corps in terms of meeting the desires or needs of local governments will be to balance the degree of flexibility necessary to meet local demands with the consistency it must have to administer a national program. Consistency is a political imperative, but good engineering or customer care may call for flexibility. These types of tradeoffs will consume the time of many policy makers within the agency over the next several years. The uncertainty over the interim will be a source of controversy. The agency's policies sometimes aren't as clear as they could be because of the highly unique technical, environmental and institutional situations presented by specific projects. In the abstract, it is difficult to describe what is and what is not an acceptable project formulation from an engineering, economic, or budget priority standpoint. Sometimes a Corps district has worked with a sponsor over a period of years under the impression that a certain policy is widely understood and accepted, only to discover that changing administration policies or budgetary priorities have soured the project's chances. Should this type of mishap continue in the cost shared future, it would contribute to a credibility problem for the Corps.

Finally, will the Senate leadership/administration cost sharing formulas work? Do they represent a realistic sharing of the investment burden? Are they high enough to provide the desired market test while simultaneously not discriminating against good projects in poorer regions? These questions certainly can't be satisfactorily answered yet, but indications to date are positive. A good reality check of the (now statutory) policies came in the Supplemental Appropriations bill passed for FY 1985. Of the 41 projects in the Supplemental, 32 local cooperation agreements (LCAs) based on H.R. 6 cost sharing percentages were successfully completed, five did not require LCAs (including four inland waterway lock and dam projects), three were selected too early in the planning process to meet the deadline, and one sponsor declined to sign the binding agreement. Since these projects were formulated under far different circumstances as far as the sponsor's share is concerned, it is reasonable to expect that alterations in the original Corps plan would be necessary to coincide with the sponsor's financial plans and capability. In fact, 12 of the 32 LCAs were negotiated for projects that constituted less than the original Corps plan. Some of these reformulations of the project plan are the first phases of a more comprehensive overall plan; six covered the initial phases of deep draft navigation projects.

Of the ten harbor projects with successful LCAs shown in Table A, sponsors signed up for an average of approximately 37 percent of the total estimated project cost. For harbor deepening projects, a "phased in" project makes good sense. The reduction of risk associated with overbuilding is a substantial benefit to the cost sharing sponsor. Once the initial benefits are realized, the project can be expanded with more certainty.

Project phasing will add to the enormous budgeting, programming and planning challenges that the Corps is going to face in the "new partnership era." New budgeting and administrative policies are now being developed. The highest budgeting priorities are being given to those projects that have signed LCAs. There is a realization at the highest levels that once these agreements are signed, the government's responsibility to deliver is increased.

Table A
Total Estimated and Initial Phase Costs of New Start Harbor Navigation Projects in the FY 86 Supplemental Appropriations Law, P.L. 99-88

Project	Estimated Project Cost (000's) Total Project	Initial Phase	Percent of Total in Loan
Mississippi River Ship Channel, Louisiana	486,000	150,000	30.8%
Mobile, Alabama	415,000	89,000	21.4%
Norfolk, Virginia	400,000	50,000	12.5%
Baltimore, Maryland	370,000	300,000	81.0%
Kill Van Kull, New York, New Jersey	290,000	145,000	50.0%
Tampa, Florida	58,000	8,000	13.8%
Subtotal	2,019,000	742,000	36
Without Baltimore	1,649,000	442,000	26
Freeport, Texas	100,000	100,000	100.0%
Sacramento, California	74,000	74,000	100.0%
Savannah, Georgia	14,000	14,000	100.0%
Jonesport, Maine	10,000	10,000	100.0%

Biodata

Mark D. Sickles

Mark Sickles has been with the U.S. Army Engineer Institute for Water Resources for a little more than two years. Over this time period he has followed the emergence of the pending federal water resources legislation as part of his assignment at the Institute. He has contributed to various studies and task forces that have tried to get a head start on implementing this historic departure from the Corps "traditional" ways of doing business. Mark has also helped organize Corps follow-up activities to the nine-month period between the passage of the FY 85 Supplemental Appropriations Act and the signing deadline for the local cooperation agreements (LCAs) which included several major port projects.

Mark received a B.S. degree in forest management from Clemson University; M.S. degree in technology and science policy, Georgia Institute of Technology; and M.S. degree in industrial management from Georgia Institute of Technology.

Impact of the New Cost-Share Regime on the Public Port Industry

R. Erik Stromberg
American Association of Port Authorities

On the assumption that we will shortly have a bill, I can characterize the impact of the pending water resources development bill in a good news/bad news context. The good news is that we will have projects after a long drought; the bad news is that the scope and timing of their construction can only be described at this point as uncertain.

As you may know, the American Association of Port Authorities, founded in 1912, today represents virtually all the public seaport authorities in the United States. The Association and our member ports have actively focused our efforts toward achievement of an omnibus water projects bill since 1981. The issues of navigation development are extremely important to U.S. ports and to those persons and entities who rely on our port facilities, obviously as well as to those who carry out those development projects. Over this past decade, the U.S. port industry has faced the growing challenge of dealing with the uncertainty of the federal government's future role in developing and maintaining the federal navigation system. Discussions over many years have brought public port administrators to the realization that development of the nation's deepdraft navigation system, with acceptable levels of maintenance, is absolutely critical to the process of port planning and development that is required to accommodate well-defined and very valid national needs.

The importance of this reality is only now at the verge of being reflective in positive legislative action. The issues involved in this legislation go to the very heart of the traditional partnership between the federal government and local project sponsors, which more often than not are public port authorities.

As you are well aware, the dredging issues addressed in the bill relate to a very important national problem that is remarkable for its complexity. These issues engage a full spectrum of economic considerations, ranging from the perceived responsibilities of users to pay for federal services, to the economic survival of a number of U.S. port communities. However, the imperative for enactment of H.R. 6 before Congress adjourns is simply absolute. We have already waited too long. If Congress goes home this week without passing H.R. 6, it could easily be 1990 before we have another good shot at an omnibus water projects bill. Neither the Corps of Engineers, the dredging industry, the public port industry, nor, most importantly, the successful conduct of U.S. waterborne commerce can afford such an additional delay. Federal navigation channel and harbor development must simply catch up with the tremendous landside facility development that has been carried out by the nation's public port authorities and private entities. Ships have been getting bigger and trade volumes have expanded yet the federal government has done very little to foster development of the federal deepdraft navigation system in the last 20 years. Congress has not produced a major water projects authorization bill since 1970. More than 40 such projects now await congressional approval. The Corps of Engineers' development and maintenance dredging projects are a fraction of what they were two decades ago.

If this bill does not pass this year, the Corps' entire channel navigation program will be in serious jeopardy. I urge you, in these last hours of the 99th Congress, to petition your congressional members to request that Rep. Rostentowski take the required steps to complete action on the legislation. The fate of the bill is in his hands. There are no more substantive issues to be resolved, and it would be a darn shame if we lost it all now.

The public port industry has accepted cost sharing, cost recovery reforms in order to move off dead center and renew our federal water resources program. As you may recall, the port industry was split, as were other industries, in the debate over how to recover federal maintenance costs. However, the industry has now united and generally supports H.R. 6 as providing the best available course to ready this nation's port system for the future demands of waterborne commerce.

I think it is important not to forget to give special recognition to Corps Secretary Dawson. We in the port industry and our allied industries have worked hard to get where we are. But I firmly believe we could not have achieved the progress we have without the tireless efforts of Bob Dawson and his team at the Corps of Engineers.

Before briefly describing the legislation, which is now at a make or break stage, let me offer a perspective, gained in working on H.R. 6, for dealing with Congress. Congress works its will at its own pace. Consider that we were potentially within hours of agreement on a water bill as part of an omnibus catch-all funding bill in the closing hours of the 98th Congress back in 1984. Now, two years later, and nearly one and a half years after the historic cost-sharing agreement between David Stockman and Republican leaders in the Senate that paved the way for congressional action on H.R. 6, we have to wait to the very end of this Congress to see if H.R. 6 can be passed and sent to the President. Perhaps such is our fate. Remember the last major omnibus water bill was passed December 31, 1970. The point is a humbling one in terms of congressional priorities. Simply put, we are not. One lesson from this is that "fixes" or follow-ups to H.R. 6 won't come easy. A second lesson is that we all need to create and increase public- and federal-level awareness of the importance of our port and navigation system to our national economy and its security.

Now, to move to the pending legislation itself. Briefly, the legislation would quote/unquote "reform" the procedures by which this nation's water resources are developed. Nonfederal sponsors of water projects, often referred to as local beneficiaries – a phrase which I believe belies the true nature and scope of those who actually benefit from the development of, in our case, our country's port and navigation system – would be required to pay a significant share of the development of federal navigation channels. The local shares are 25 percent of total project cost for projects 20 feet to 45 feet, with 10 percent of the project cost to be paid to the federal government over a 30-year period. For projects deeper than 45 feet, local sponsors would have to pay 50 percent of total project costs, again with the 10 percent payback. The payback in both cases could be directly offset, dollar for dollar, by traditional local expenditures for lands, easements, rights of way, dredge spoil disposal and utility relocations.

The bill would authorize construction or study of 262 new Army Corps of Engineers water projects – 41 port, seven inland waterway, 113 flood control, 24 shoreline protection and 77 water resources conservation and development projects. In addition, the bill would authorize 31 studies, 73 project modifications and 63 other miscellaneous projects and programs.

The bill authorizes a total of $16.3 billion in spending, of which $12 billion would be paid by the federal government and $4.3 billion by nonfederal interests such as states, localities, port authorities and commercial navigation companies.

The House version of H.R. 6, approved last November 13, would have authorized 316 projects and a number of major new programs at a total cost of $20.8 billion. The Senate bill, passed March 26, contained 191 project authorizations and had a total cost of $12.9 billion. The conference began June 5.

I should remind you that authorization by H.R. 6 only makes a project eligible for funding; the measure does not appropriate any money. It does, however, limit how much the Corps can spend on construction in each of the next five years. It imposes ceilings on total construction obligations that range from $1.4 billion in fiscal 1987 to $1.8 billion in FY 1991. (The Corps received $1.5 billion in total construction appropriations in FY 1986.)

Projects in H.R. 6 that do not receive construction funding within five years would be automatically deauthorized. The measure deauthorizes outright 293 old, unfunded Corps projects that have an estimated total cost of $11.3 billion.

About 100 of the projects in the conference agreement have not completed the Corps' evaluation and planning process. Nearly all of them came from the House version of H.R. 6, and Senate conferees agreed to include them in the conference report with conditional authorizations. In most cases, the bill would fully authorize such projects but make their construction contingent on completion of their Corps' evaluation studies and approval by the secretary of the Army.

There are, not surprisingly, a host of other provisions in this legislation that will result in basic changes in the funding and procedures involved in channel improvement projects. I will not enumerate them all here. I would like to point out that under this legislation, the Corps of Engineers and the local project sponsors will be entering into a new era, a partnership, not a supplier/customer relationship, but a partnership through which hopefully the Corps and the ports can resurrect and energize the development of our country's port and navigation system. In this light, we urge the Corps to develop mutually satisfactory standardized contractual terms and conditions for channel projects in recognition of the fact that the local sponsor now will have an increased role in project decisions. In this new cost-sharing environment, the local sponsors will have rights as well as responsibilities, and the Corps will have a responsibility to the local sponsor.

To answer what the new legislation will mean to our industries, I must return to what I said at the outset of my remarks. While there will be projects coming on stream, their timing and scope remains uncertain. For example, we have nine deep draft projects funded for construction by the 1985 supplemental appropriations bill, and the new cost-sharing provisions have already had an effect. One of the original projects was withdrawn, and four others were downsized in scope. Projects authorized for construction under H.R. 6 will depend on adequate Corps appropriations by the Congress to come on stream, but they will also be dependent upon the ability of the local sponsors to finance their share. In this context, the legislation falls far short of providing the ports with sufficient funding mechanisms, which compounds the uncertainty of future public port authority initiated channel development projects. It should be noted that our industry is currently experiencing pressures to rationalize capacity and scale back or more narrowly focus future development plans. Based on discussions with ports around the country and the Corps of Engineers, I would estimate we should see no more than eight to 10 projects ready for construction during any given year. While most of the projects authorized by H.R. 6 will move forward, perhaps as many as a quarter will be dropped or significantly downsized.

The next generation of channel development projects will likely be even further diminished in number, though the projects may be bigger. Once we have reduced the backlog of projects, there will be less pressure for Congress to move ahead with new authorizations. Perhaps the new authorization cycle will be four to six years instead of two years as it was in the past before 1970. The port industry will have to be exceedingly aware of the financial bottom line before committing to new channel development. Other factors, such as availability and cost of dredge disposal sites, must be considered.

In conclusion, we are on the verge of a new era in the development of our nation's port system. The challenges will be serious, but I can assure you of the commitment of the public port industry to our successful future. Thank you.

Biodata

R. Erik Stromberg

On February 1, 1985, Erik Stromberg was appointed director of governmental relations for the American Association of Port Authorities. He was promoted to vice president of governmental relations on May 1, 1986. On June 2, he became acting chief executive officer of AAPA until a successor to Ron Brinson is named.

Mr. Stromberg joined AAPA after having served, since April 1983, as a policy analyst in the Office of Policy Planning and International Affairs at the Federal Maritime Commission. At the FMC, he specialized in port and intermodal regulatory policies. Previously, he worked as a legislative assistant and marine policy specialist for U.S. Representative Glenn Anderson of Los Angeles, California, chairman of the House Public Works Surface Transportation Subcommittee and ranking majority member of the Merchant Marine Subcommittee.

Mr. Stromberg holds a B.S. degree in political science and an M.S. degree in marine affairs from the University of Washington. He began his work in Washington, D.C., in 1982 on a year-long Congressional Sea Grant Fellowship. During his graduate studies at the Institute for Marine Studies, his areas of concentration were port management, public enterprise theory and marine transportation. He was awarded the McKernan Prize, an annual award by the Institute for the outstanding master's thesis. Mr. Stromberg's thesis addressed the political and economic basis for public port investment strategies.

An Industry Perspective on Cost Sharing Legislation: Are We Getting Well?

Paul R. (Rich) Dickinson
National Association of Dredging Contractors

Good afternoon. I am Rich Dickinson, secretary-treasurer of the National Association of Dredging Contractors. I am substituting for Jack Downs, president of the Association, who regrettably is not able to be here today. I am sure you were all anxious to hear Jack but I will do my best to fill in, a no small chore.

The subject for this presentation, which I should tell you was selected by others than Jack or I, is of an interesting nature. I will assume that it does not infer that the industry should have a position on the merits of cost-sharing vs. federal funding. The improvement and maintenance of the country's harbors and channels have traditionally been funded from tax revenues. I believe that the cost-sharing concept by which a substantial portion of this cost should be borne directly by those deriving primary benefit was first espoused by the Carter Administration and wholeheartedly promoted by the Reagan White House.

Certainly local participation in project costs will require very close study of the cost-benefit ratio because local funding will generally be by means of debt securities which will have to be repaid. This fact should allay the long-time hue and cry that water projects are "pork barrel politics."

On the other hand, we sincerely hope that local participation will not result in a loss of control of the program by the Corps of Engineers. Their experience in all aspects of this vital segment of the country's economy has been accumulated over almost 200 years and while we have our occasional differences with them, we commend them as a devoted and very professional organization.

I am not sure if the assigned subject title infers comments from our industry regarding the preferred method of project funding – local participation versus federal appropriation. If such is the intent, I will have to invoke the much-used phrase "No Comment." We feel it wholly improper for a contractor to take a position on how the funds to pay him for his efforts are generated. This decision on a funding vehicle must be left to greater minds than ours.

The last phrase in today's subject – "Are We Getting Well" – is interesting. It is implicitly in the words "getting well" that the patient has been less than healthy. In the case of the dredging industry this is certainly the case. In the recent past I can think of 13 dredging contractors who have left the industry. Atlantic Gulf & Pacific, Gahagan, Williams-McWilliams, Jahncke, Merritt Chapman Scott, Standard, Bauer, New England, Henry du Bois, Radcliff, Hydraulic Dredging, Fitzsimons & Connell, and Aprundel were all major forces in the field who have either ceased dredging or gone out of business entirely. Conversely, I can't think of any new entrants to the business.

During the same period, however, those still in the business, recognizing the eventual necessity for the United States to catch up with the other maritime countries of the world, have made remarkable strides in expenditures to provide their customers with the most modern dredging fleets in the world. This has been accomplished despite reduced expenditures for their services. Since World War II, cutter dredge power has increased tenfold, clamshell bucket maximum size has gone from 14 to 50 cubic yards and dippers from 12 to 25 cubic yards. Dump barges have gone from 1,000 cubic yard capacity to 6,000 and towing tugs from 1,000 horsepower to 5,600. Rock drilling and blasting methods have been improved immeasurably.

The advent of electronics has revolutionized methods of sounding and dredge positioning in both speed and accuracy.

The work I used to do with a survey crew in a week when I was a field engineer is now done in minutes. Instead of the shale boat, tag line, sounding lead line and wooden range buoys, today's field engineer uses a $200,000 high speed launch and state-of-the-art electronics. The old time methods of plotting cross sections and computation of quantities are now done simultaneously by computer. Dredges can be positioned with pinpoint accuracy even though 20 miles offshore. These improvements have been brought about by much experimentation and expenditure of money.

Without a doubt, however, the most outstanding achievements by the dredging industry have been in hopper dredging. An activity solely done by the Corps of Engineers 10 short years ago now is currently engaged in by six contractors operating 14 of the most modern units in the world. Again, this has not come about easily. This fleet represents a capital investment of over $300 million and provides the United States with more than sufficient hopper dredging capability on all of its four coasts – Atlantic, Gulf of Mexico, Pacific and the Great Lakes. I should point out that this has been accomplished without the subsidies enjoyed by most of our foreign competition.

Let's look at the make-up of our industry for a minute. Compared to other American industries, dredging is very small. Total revenues are estimated to be about $1 billion – in a good year. The industry consists of many small ma-and-pa operators who are generally provincial. The largest company has only about 12 percent to 15 percent of the market and only one or two work on all four sea coasts. Over 100 contractors submit bids on Corps dredging contracts annually.

In addition to the 14 hoppers, the dredging industry fleet is comprised of over 100 cutterhead dredges, 18 inches and larger, 80 bucket dredges of 6 cubic yards and larger, and one 38-inch dust pan. In addition to the $300 million expended for the hoppers, dredge owners have spent over $200 million on new, up-to-date cutters, buckets and the dust pan. All the new units and a number of others are certified by the Coast Guard to work in unprotected open waters – the area where a substantial amount of the cost sharing work will occur.

The industry is currently operating at less than 50 percent utilization and its capabilities so impressed the Corps that Lt. Gen. E.R. Heiberg, chief of engineers, testified before a congressional committee several years ago that the nation's private dredging capabilities were such that four major deepening projects could be done simultaneously – a work load substantially beyond reasonable expectations. Since that time the industry has further improved and added to its fleet providing additional capability.

The industry has an enviable record on overseas work. Despite the tax advantages enjoyed by a foreign competition, United States dredging contractors worked successfully in the Middle East, Africa, South America, Central America, the Caribbean and Canada. Many of these overseas projects involved operation miles offshore.

In addition to navigational work, the industry, often working closely with the Corps, has been instrumental in other applications of the dredging process. Miles of beach have been restored, landfills for residential and commercial development created, underwater berm-like mounds built to protect shorelines and concrete aggregates mined. The Corps, with the cooperation of the industry, very recently conducted a workshop in Pensacola, Florida, to acquaint interested parties in beneficial usages of dredged material.

Dredging has recently gone through a difficult period during which all dredged material was automatically considered to be polluting the environment. Again, with the active cooperation of the industry, the Corps conducted a multi-year testing period at its Waterways Experiment Station at Vicksburg, Mississippi. These tests proved that in reality, less than five percent of dredged material is polluted. After identifying the problem materials, improved permit procedures were instituted to provide protection for the environment.

We are very proud of the Corps of Engineers' reserve fleet (CERF) of privately owned dredges which can be ordered to do emergency and national defense projects on short notice. This fleet has been activated a number of time in paper exercises and once an industry unit actually was ordered to Mobile, Alabama, to provide emergency dredging. Results of all these exercises proved the workability of the program. Procedures are such that the CERF can be mobilized for overseas use as well as for the continental United States. Unlike similar programs conducted by the Air Force and the Navy, there is no cost to the government until mobilization actually occurs.

The relative sizes of the Corps of Engineers dredge fleet and that of private industry was debated for years. Traditionally, the Corps was the industry's largest competitor as well as its biggest customer. As late as 1948 the Corps owned and operated over 140 dredges. Most of this fleet was antiquated and the industry contended that rather than spending federal funds to modernize it, given the opportunity to bid on work then set aside for the Corps dredges, the industry could prove itself able to do the work in a timely manner and at competitive prices. This opportunity was offered to the private owners in the industry capability program started in 1976.

During the over four years of the program, the industry proved its contentions. As we all remember, Public Law 95-269 was signed by President Carter. It created the "minimum fleet" of Corps dredges, but the actual number of units was not determined until after President Reagan was in office.

Until the 1970s, Congress normally passed a river and harbor improvement bill every year or two. In the last decade, however, no significant improvements have been approved. Deep port legislation is long overdue. In addition to the $3 billion to $4 billion in improvements required during the span of a decade, a currently undetermined increase in annual maintenance workload will result.

We in the dredging industry look forward to the challenges afforded us by the cost-sharing legislation. While I may have bored you by much of the previous portions of this discussion, they were put forward to point out that the industry worked hard and spend freely to prepare itself for this opportunity. We are ready, willing and able to provide the United States with a port and channel system comparable to, or better than, any other maritime country in the world. All we ask is the chance.

Whether we get well in the process is difficult to predict. I hope that you will agree with me that the patient which has experienced a long malaise is entitled to restoration to good health.

Biodata

Paul R. (Rich) Dickinson

Mr. Dickinson is a graduate of Cornell University, Ithaca, New York, with a bachelor of civil engineering. He is a member of Tau Beta Pi and Chi Epsilon. He served as a communications officer in the U.S. Maritime Service during World War II and as an officer in the U.S. Army Corps of Engineers during the Korean War. Mr. Dickinson spent one year with the 13th Engineer Combat Engineers, 7th Infantry Division, in Korea. He joined Great Lakes Dredge and Dock Company after college and worked as a timekeeper, field engineer, project superintendent, estimator, corporate secretary, vice president and a member of the board of directors.

Mr. Dickinson retired from Great Lakes in 1986 but continues to serve as secretary-treasurer of the National Association of Dredging Contractors, Washington, D.C.

Widening The Gaillard Cut

Guillermo Van Hoorde, Jr.
Chief, Canal Improvements Division
Office of the Administrator (EP)

Introduction

Organization

The Panama Canal just celebrated its seventy-second year of service to world shipping. Since August 15, 1914, when its locks were first opened to the maritime community, continuous maintenance and improvement programs have enabled the canal to keep pace with the changing demands of world trade.

The canal organization changed radically as of October 1, 1979 with the implementation of the Panama Canal Treaty. Prior to that date, it consisted of the Panama Canal Company and the Canal Zone Government.The Canal Zone Government administered the various civil functions such as: schools, hospitals, fire and police protection. The company side included the commercial-type operations such as ports, the railroad, and commissaries, as well as the basic marine and engineering functions. There were six operating bureaus under the combined company/government and the administration of the organization was in the hands of a governor/president.

Some functions such as health services and educational services were transferred to the Department of Defense. Others such as customs and immigration, some fire protection, the Panama Railroad, and the ports were transferred to the Republic of Panama.

The Panama Canal Commission (which up to the year 2000 will continue to be a United States government agency) has been streamlined to three major bureaus assisted by staff units to carry its mission to operate and maintain the Canal. The Marine Bureau administers the navigation through the canal and the operation of the locks. The Engineering and Construction Bureau is in charge of construction, dredging and maintenance of facilities.

The General Services bureau provides protection, storehouse, and transportation while taking care of community services. (These services however are being phased out). Staff units are responsible for financial management, planning, personnel administration, counsel, and public relations.

We just completed our seventh year of operation since the implementation of the treaty between the United States and the Republic of Panama. During this time, that Canal has provided uninterrupted, efficient transit service to international shipping and has moved ahead with necessary capacity improvements and training programs to ensure the future efficiency of the Panama Canal: all, while meeting a basic requirement to operate on a break-even financial basis.

Budget

The total operating revenues amounting to approximately $430 million is mostly generated from tolls (Figure 1). Other revenues like tug and launch service, electric power, and water account for the remaining 25%. On the expense side the two large ticket items are operations and maintenance. Payments to Panama include $0.33 per PCC ton passing through the canals specified in the 1979 treaties. Support services include items such as power, communication, water and storehouse facilities. Payments to the United States include interest on the investment and early retirement amortization. Other costs include items like the cost of operating the staff units and the cost of the General Services Bureau.

Commodities

The great majority of tonnage through the Canal is bulk-type cargo with over half of the total cargo concentrated in four product groups: petroleum, grains, phosphates and coal. Container cargo, however, is becoming increasingly important. Last year, more than 11.0% of our business, some 15 million tons, was containerized.

Vessels transiting the Panama Canal move cargo over a number of major world trade routes. The dominant route for the last several decades has been between the east coast of the United States and the Far East - principally Japan.

Figure 1
FY 1986 Budget

	Millions
Operating Revenues	430
Tolls	315
Others	115
Operating Expenses	430
Operations	115
Maintenance	85
Payments to Panama	75
Support Services	70
Payments to U.S.	30
Others	55

Transits Pattern

Since the official date of its opening, more than 650,000 vessels of all types have passed through the waterway. Commercial vessels have been the dominant element in the flow of transits, with that dominance challenged only in periods of war or other conflict by U.S. government traffic.

Commercial traffic grew steadily after a shaky start in 1915 until the Great Depression. It never really resumed its growth until after World War II, a time when Canal traffic was dominated by U.S. military traffic. A strong growth pattern began in the late 1940s and continued with relatively minor ups and downs until the mid-1970s when we experienced a major recession combined with the opening of the Suez Canal. Transits rose considerably from 1977 through 1982 mainly due to the Alaskan oil trade, but with the loss of that trade to the Trans-Panama Oil Pipeline, transits fell sharply. The decline in the number of ships has been offset, however, by dramatic increases in average vessel size.

Gaillard Cut Restrictions

The average size of transiting vessels, as measure by Panama Canal Net Tons, was relatively stable until the early 1950s when larger specialized vessels began to appear in our trades, reflecting the need to move increasing amounts of raw materials and other commodities in the most efficient way possible. Between 1955 and 1985, the average size of vessels using the canal increased by a factor of three.

Most importantly, the number of the largest vessels the Canal can accommodate — those of 100-foot beam and over — has increased sharply in recent years. The number of vessels in that category have increased from 115 or less than 1% of total transits in 1965 to nearly 2,400 or 20 percent in 1986. Our most recent forecast indicates the likelihood that sizes of vessels will continue to increase in vessel size will be sufficient to capture the 2 percent increase in tonnage expected in the future.

Except for the locks structure, the narrowest section of the Panama Canal is an eight-mile stretch called Gaillard Cut. Because of its narrow 500-foot width, transit is restricted in this area of the Canal.

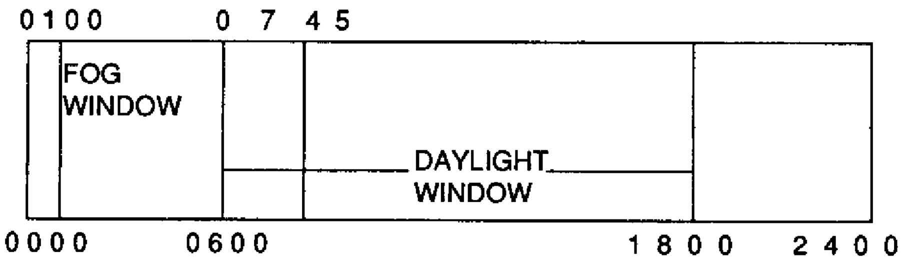
The existing restrictions in the Gaillard Cut are shown in Figure 2. Vessels with beam smaller than 80 feet can transit at any time and can meet traffic in opposite direction. Vessels in the 80 to 90 beam size category are usually unrestricted unless their draft is 36 feet or over, in which case they must transit Gaillard Cut during daylight. Vessels 91 to 95 foot beam must transit during daylight and in addition cannot meet vessels navigating in opposite direction if their combined beams exceed 170 feet. And finally, vessels 95 feet and over are restricted to daylight and clearcut transit.

The window available for daylight vessels (Figure 3) that normally would extend from 0600 hours to 1800 hours is frequently reduced by fog. Indeed, during the nine months of rainy season, transit schedulers automatically assume that traffic in Gaillard Cut will not begin until 0745 hours, when fog lifts. These daylight and one-way traffic restrictions limit the number of vessels that can be processed per day. Management recognized the need to study the one-way traffic restriction problem, and in 1983 the Board of Directors approved the funds required for the study.

Transit Restrictions
Figure 2

Beam Size	Restriction
Below 80'	None
80' – 90' Draft >36'	Daylight
91' – 95'	Partial Clearcut Daylight
Over 95'	Clearcut Daylight

Impact of Transit Restrictions
Figure 3



Gaillard Cut Study

Objective of Study

The cut widening study is aimed at determining the need for widening, the dimensions to which the cut must be widened to permit two-way traffic of the largest size vessels that can use the Panama Canal, and the benefits to be derived from widening. The specification of the optimum channel would result from a reasonable balance between the cost of the excavation and the degree of safety that would be afforded transiting vessels.

To carry the study to its successful completion, operational, technical, environmental, economic and financial considerations had to be evaluated.

Operational Analysis

Objective — The operational analysis establishes when the project is needed, how the project should be executed, and how resources will be affected. The need for the project should be executed, and how resources will be affected. The need for the project is determined by the interaction of traffic (in number and size of vessels), capacity, quality of service as measured by the time spent in Canal waters, and safety. Whenever the capacity of the Canal is not increased to match higher traffic, vessels spend more time being processed. Delays, as much as tolls, are perceived by customers as a cost of using the Canal. If delays are excessive the cost would go up and some traffic would be diverted to alternate routes.

Traffic Forecast — The operational analysis required a forecast of arrivals broken down by size of vessel. The forecast prepared by Manalytics, Inc., shows a substantial increase in the number of restricted vessels - those requiring either daylight or clearcut transit. Table 1 shows that their number will almost double in the next 25 years.

Arrivals were used in the development of hypothetical transit schedules which simulate the process followed in the real-life scheduling of vessels. Hypothetical schedules proved an effective and efficient tool to reproduce real-life scenarios. With the hypothetical schedules the commission was able to determine future operating

Traffic Forecast
Table 1

Beam Size	1985	1995	2000	2010
Unrestricted	19	14	12	9
Below 80	14	10	8	6
80-90	5	4	4	3
Restricted	12	16	18	22
80-90	1	1	1	1
91-95	2	2	2	2
96-100	2	2	2	2
Over 100	7	11	13	17
Total	31	30	30	31

revenues and cost, as well as the resources information required for the economic analysis of the project.

Canal Waters Time — The operational analysis draws on "queuing theory," whereby customers arrive randomly at a service facility, await service, and depart after service is provided. The capacity available at the service facility (the canal in this case must exceed the average rate of arrivals in order to maintain customer waiting time within acceptable limits.

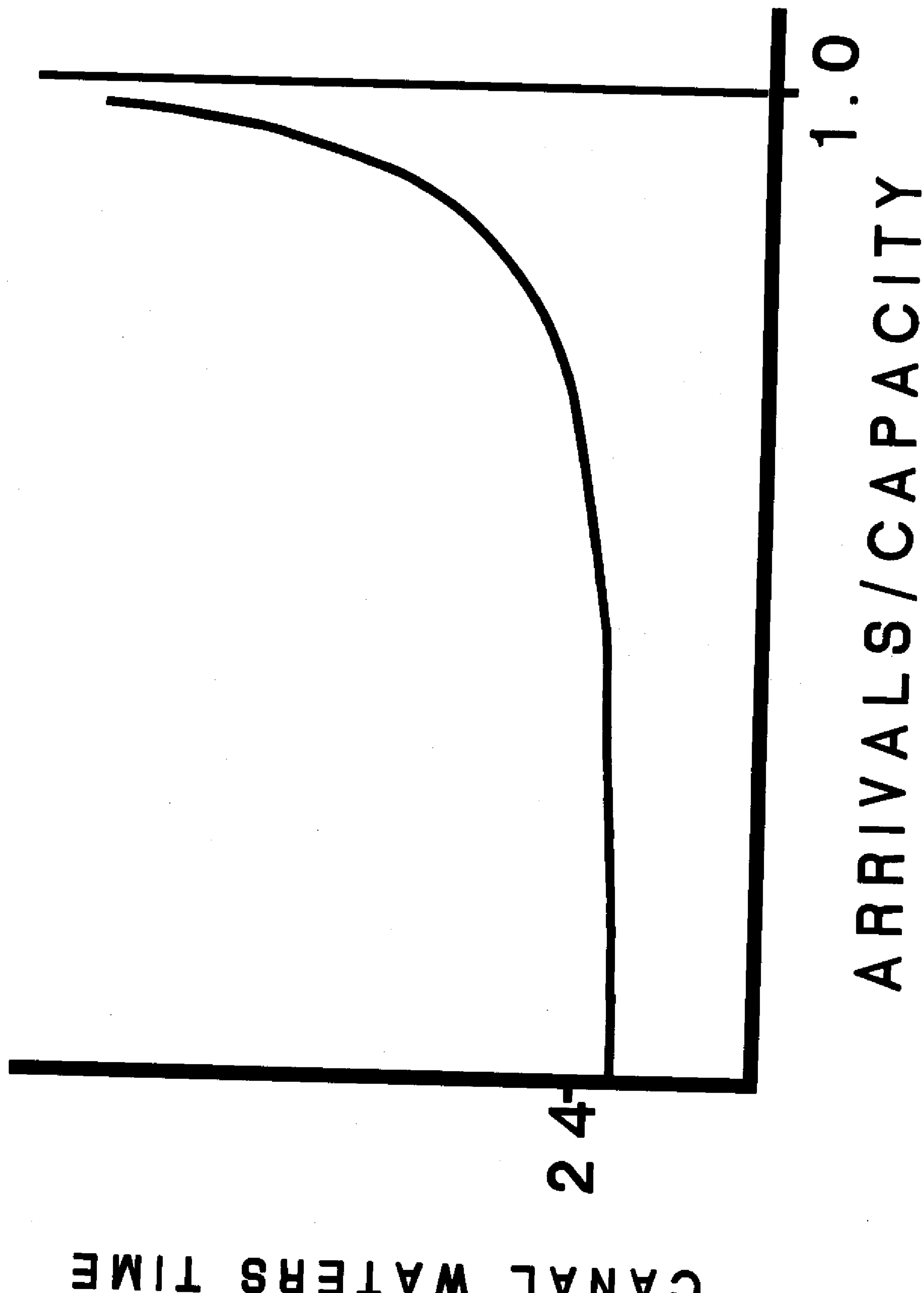
Figure 4 is representative of the conditions at the Panama Canal. The time spent in Canal waters, or Canal Waters Time as it is called, would be uncontrollable when arrivals approach capacity. At a lower level of arrivals CWT can be maintained around 24 hours which historically has been used as the standard for quality of service.

A computer model was developed to determine the behavior of CWT capacity at different levels of arrivals and capacity. The model's particular value was in evaluating the ways CWT deteriorates as arrivals approach maximum capacity of the Canal, and the behavior of CWT as result of a lock lane taken out of service for periodic maintenance and overhaul.

Results (Table 2) show that without widening the levels of CWT during lock lane outages would increase to unacceptable levels. At present, CWT is maintained at around 24 hours for 323 days out of the year. By the year 2005 CWT would increase to 28 hours during 172 days and to 82 hours during 194 days.

Canal Waters Time
Table 2

Year	Normal Operation		During Lane Outages	
	CWT	Days	CWT	Days
1986	24	323	31	42
2005	28	172	82	194


ARRIVALS/CAPACITY
1.0
24
CANAL WATERS TIME

The study shows that without widening the Canal can continue to operate with acceptable quality of service up to around 1997 when 17 daylight vessels are expected to arrive per day. When arrivals exceed 17 daylight vessels, the CWT is expected to deteriorate. After widening, the Canal will be able to process the traffic volumes expected during the next 50 years with reasonable expediency and safety.

Finally, the operational analysis also identified the resources required and the optimal excavation sequence.

Alternatives to Full Widening — Several alternatives to full widening were considered (Figure 5). Some, like the construction of tie-up stations, fail to provide adequate solution to the heavy traffic of daylight vessels expected. At most these alternatives provide theoretical gains of only one to two vessels and require a synchronization of traffic unlikely to be achieved. Other alternatives like partial full widening are nothing more than breaking down the widening project in phases.

Technical Analysis

Objective —The purpose of the technical analysis is to determine the optimum channel; slope design and excavation volume; and construction methodology and cost. The channel design work was performed with the assistance of the Maritime Administration's Computer Aided Operations Research Facility (CAORF) at Kings Point, New York. The slope design and determination of excavation volume were done internally by the commission's own geotechnical engineers, and the excavation methodology and cost was contracted out to the United States Army Corps of Engineers.

The three elements of the technical analysis are shown graphically in Figure 6. The width of the channel from the new prism line and the slope of the bank determine the volume of material to be excavated. The corps of engineers considered both volume and nature of material to arrive at the optimal excavation methodology and cost.

Channel Design

Objective — The objective of the CAORF study is the establishment of the channel design that requires the least amount of excavation, while providing two large Panamax vessels the same or greater safety that is afforded now to the largest vessels that are allowed to meeting in the 500-foot wide channel.

Steering Quality Profile - To quantify the safety achieved during a meeting encounter, a multi-dimensional performance measure referred to as the steering quality profile (SQP) was developed for this project. The measure consists of four independent indices, each of which addresses a different aspect of shiphandling.

• The *relative clearance margin* evaluates proximity to obstacles. A perfect score of 1.0 is given if the pilot exactly splits the available lane between the bank and the traffic ship.

•The *control force margin* evaluates available control reserve during the meeting. If no control effort is expended, a perfect score of 1.0 is given.

• *Course changing quality* or its reciprocal, yaw rate variance, evaluates directional control.

•The *subjective measures* evaluate the degree of difficulty reported by the pilot in completing the encounter.

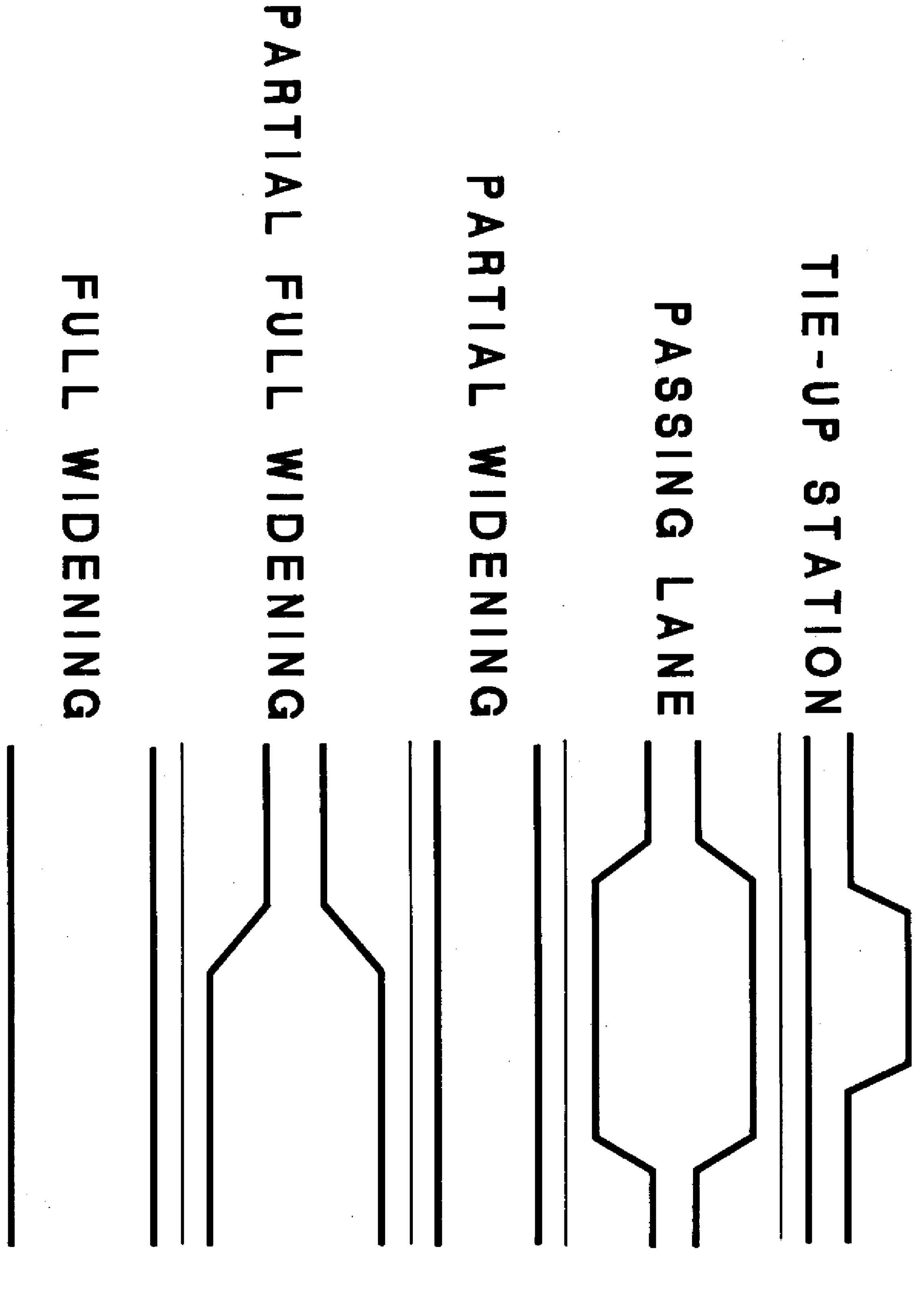
The steering quality profile was measured using a validation vessel to establish the standard quality of navigation that had to be met with the design vessel in the widened channel.

Model Development — The validation vessel was defined as an 85-foot beam generic bulk carrier vessel. The design vessel was defined as a 106-foot beam bulk carrier. The behavior of ship models was determined using specialized laboratories capable of measuring hydrodynamic forces acting on vessel navigation in restricted channels, by recording data of vessels actually navigating Gaillard Cut, and by observing behavior in computerized simulators capable of reproducing navigational characteristics. The SSPA Laboratories were used to establish hydrodynamic forces, while CAORF laboratories were used for simulation using PCC pilots.

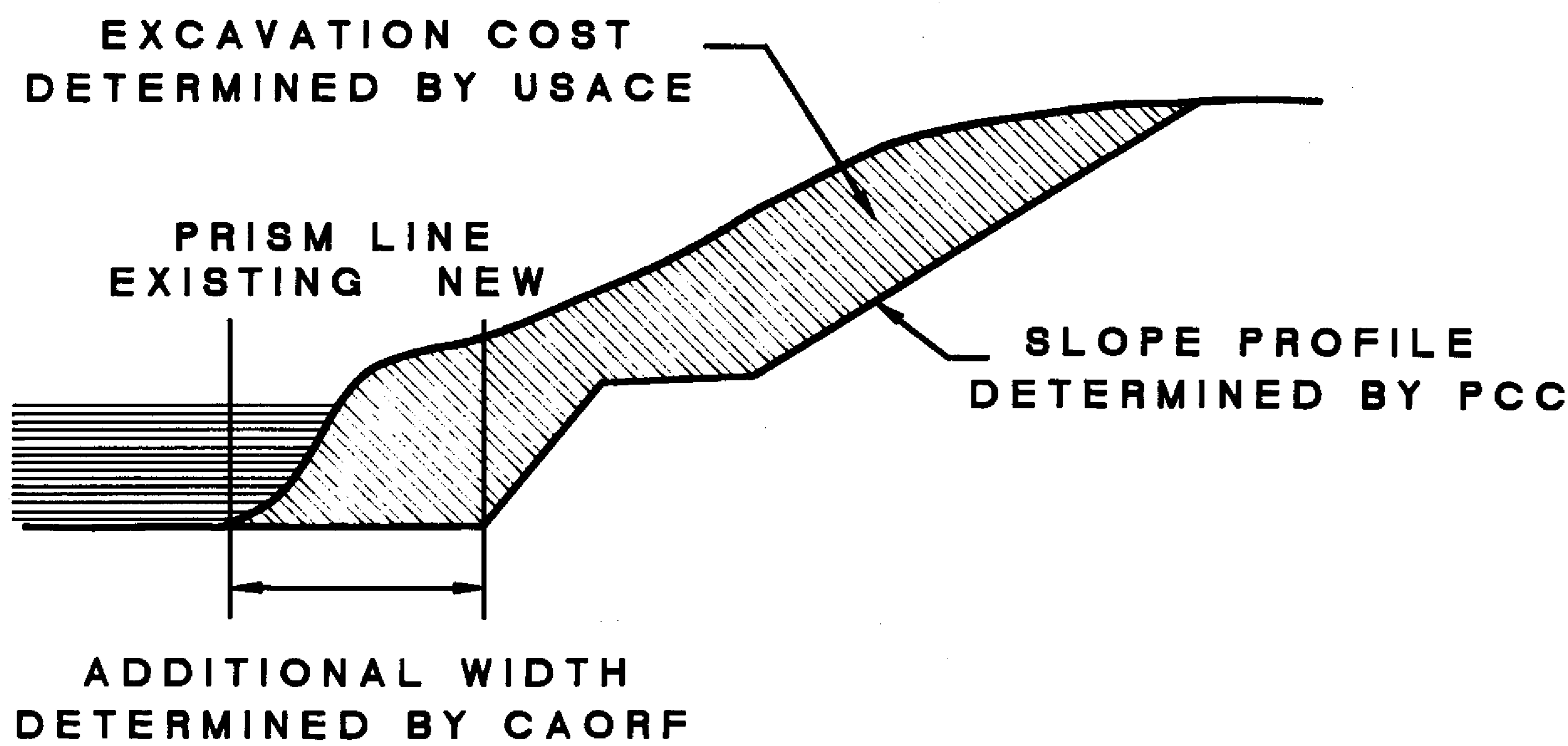
Compressed Time -- Because of the large number of possible layouts that had to be tested, compressed time analysis using automatic pilot logic was used. A layout requiring little excavation was tested under different operational conditions for Steering Quality Index. If results fell short of the criteria established for acceptance, a new layout requiring more excavation was tested. The process was reiterated until the navigational criteria was met. That layout was subsequently validated by commission pilots using real-time simulation (Figure 7).

Recommended Channel -- The process ended with the identification of dimensions which met the safety requirements and provided for the least amount of excavation. That necessitates widening straight sections from 500 to 630 feet and excavating the inside of curved sections to a width up to 730 feet, while increasing the radius of curvature up to 300 feet. The study also concluded that the transition from straight to curve sections required flaring at a 20 degree angle (Figure 8).

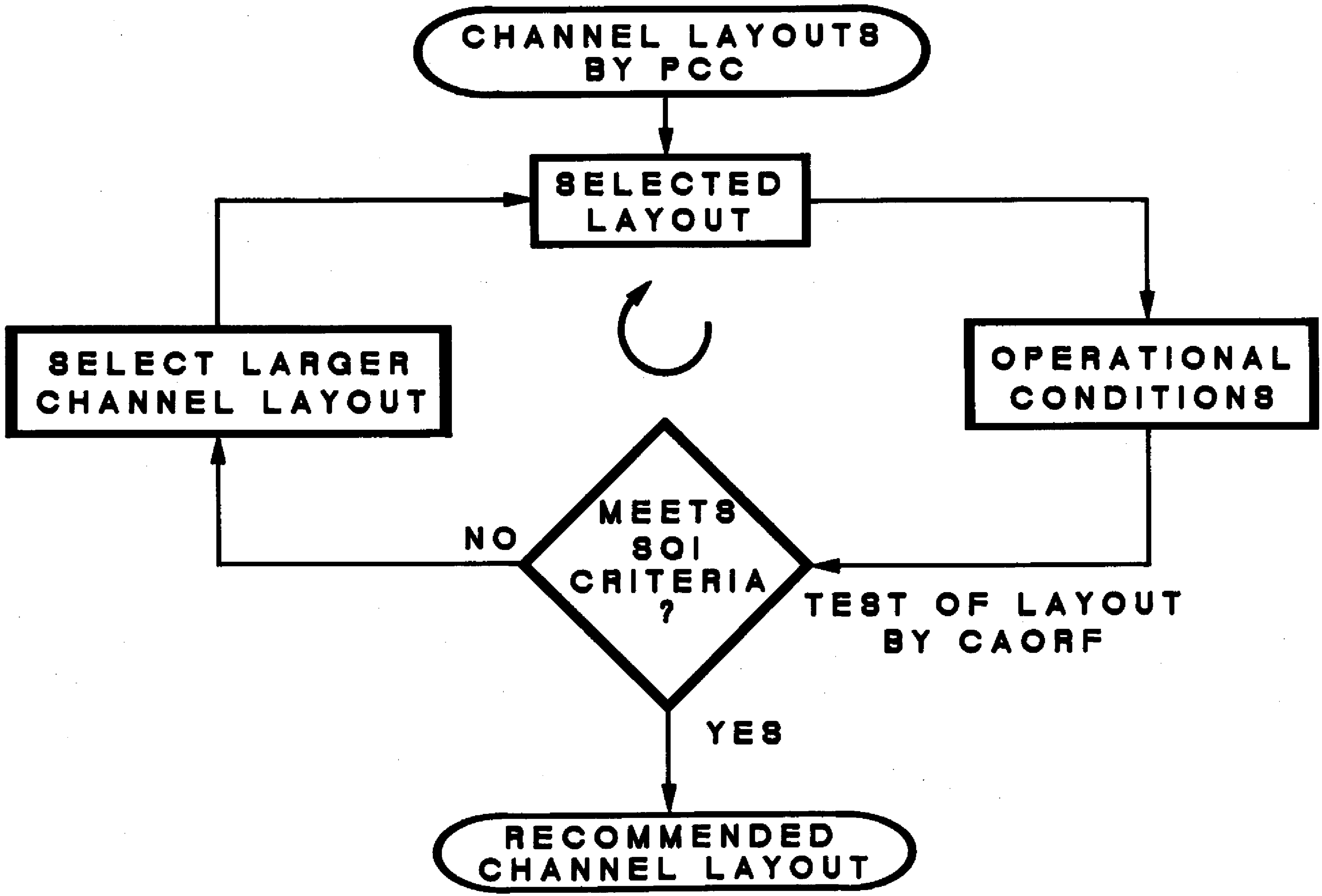
ALTERNATIVES



TECHNICAL STUDY



COMPRESSED-TIME ANALYSIS STRATEGY



Recommended Channel Dimensions
Figure 8

Section	Existing	Recommended
Straight	500'	630'
Curves:		
Width	500'	Up to 730'
Curvature		Increased up to 3000'
Transition	None	20%

Geotechnical Effort

Objective — The objectives of the geotechnical effort are the design of the slopes of the channel and the computation of the corresponding excavation volume. In the design of the slopes one of the controlling factors is the concern for landslides.

Optimization of Slope Design — Slopes have to be designed to achieve the additional channel width at the lowest overall cost. The lowest overall cost of the excavation work implies a compromise between the initial construction expenditures and the long-term maintenance requirement (Figure 9).

Material Types — To design stable slopes particular consideration must be devoted to the strength of the materials along the shearing surfaces. If determining the strength properties of a natural slope is difficult, the task of assessing the shear strength characteristics of the many materials in Gaillard Cut becomes a true challenge. Consulting services, including the United States Army Corp of Engineers, provided guidance on this subject. Significant progress has been made in estimating strengths of the materials for the stability analysis and slope design. Research conducted in the Canal's Soils Laboratory, in conjunction with the feasibility study, has contributed to this goal.

Volumes — The volume estimates obtained for the recommended channel and for the present slope designs was the basis for the project cost estimate. Results are presented in Figure 10 by sectors.

Volumes
Figure 10

	In Million Cubic Yards			
	North	**Central**	**South**	**Total**
Dry Soil	0.8	5.6	2.1	8.5
Dry Rock	1.6	7.0	5.4	14.0
Dredging	3.4	5.0	3.2	11.6
Totals	5.8	17.6	10.7	34.1

Excavation analysis

Objective — The Gaillard Cut project consists of the removal and disposal of approximately 34 million cubic yards of earth and rock in order to widen the Gaillard Cut. The United States Army Corps of Engineers was tasked to establish optimum construction methodology and cost.

Optimum Methodology (Figure 11) — The preparation of the area includes clearing, grubbing, ripping, and construction and maintenance of access and haul roads.

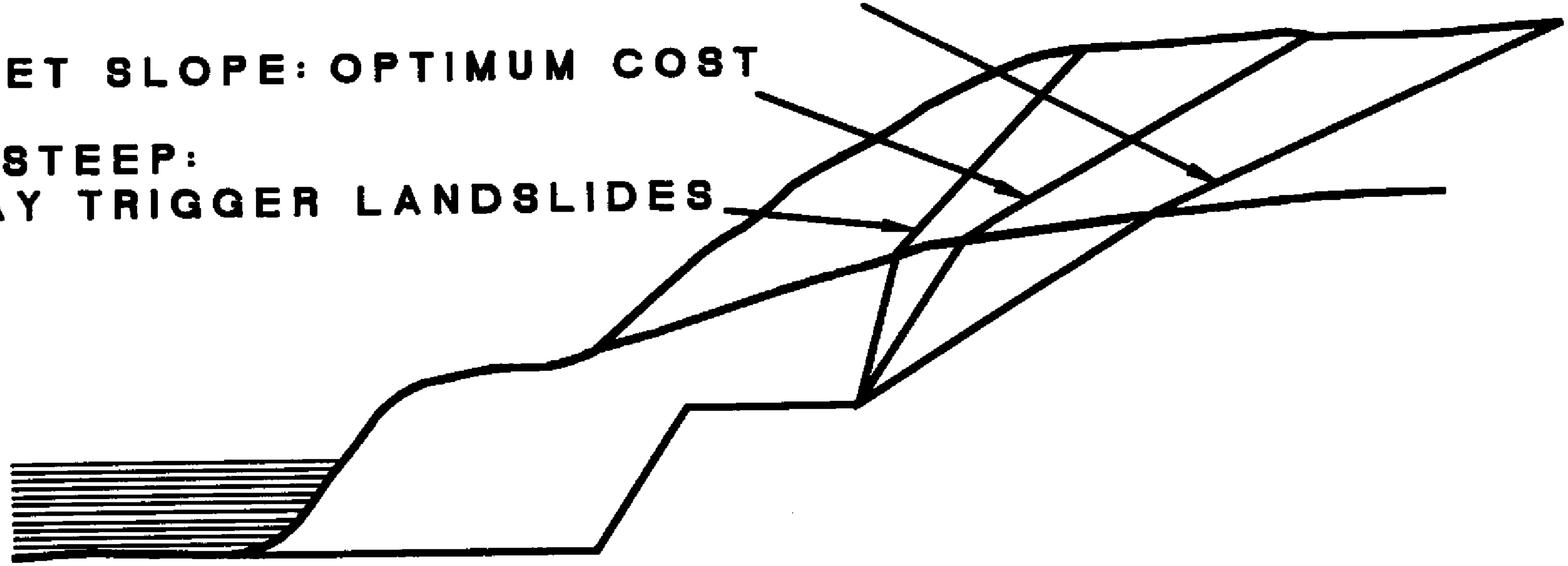
A significant portion of the "dry" material lying above the top of sound rock consists of consolidated material and weathered rock which will not require drilling and blasting but will need to be ripped to facilitate loading and hauling. Materials can be loaded and haled effectively using scrapers assisted by push-tractors. This method is particularly well-suited for relatively short haul distances over moderately-steep graded routes such as will be the case in Gaillard Project.

OPTIMIZATION OF SLOPE PROFILE

TOO GRADUAL: ADDS EXCESSIVE EXCAVATION

TARGET SLOPE: OPTIMUM COST

TOO STEEP:
MAY TRIGGER LANDSLIDES



All sound rock will require breaking up to enable its excavation. To accomplish this, drilling and blasting is considered the most practical, conventional method suited to the conditions of this project. The material can then be effectively loaded by large rubber-tired front-end loaders and hauled in large dump trucks. Spreads comprised of this equipment are very versatile, mobile and readily adjustable to compensate for variations in material characteristics, haul distances, and the like.

The wet material can be excavated by dipper dredge and transported in bottom-dump scows to disposal areas in Gatun Lake. This appears to be the most effective method for accomplishing the "wet" work, since this material is generally inaccessible to land-based equipment. Dipper dredges are better suited than most other equipment for "sweeping" to clean up the bottom of a cut.

Excavation and disposal of material from Gold Hill presents particular problems primarily due to access. There are unstable areas on upper and lower sides of the hill where major slides have occurred. The sides of Gold Hill from which the material slid are virtually vertical, preventing access by sidehill haul roads in the normal manner. No firm conclusions could be reached as to suitability and practicality of the exotic methods considered for the additional excavation at Gold Hill but a contingency estimate of cost has been included.

In contrast to the aforementioned methods selected, other excavation methods were not deemed practical or cost effective. Excavating with large dragline, clamshell or backhoe was considered inferior to the dipper dredge because of project magnitude and existing conditions. Excavation with cutter-head pipeline dredge and pumping to upland disposal areas was considered impractical when compared with the range of other alternatives. Considerable delays might be incurred where large boulders and lenses of hard material are anticipated to impede operations or even damage the pipeline dredge. Excavating with wheel excavators and transporting with conveyors were not considered suitable because of the relatively long distance the material has to be transported.

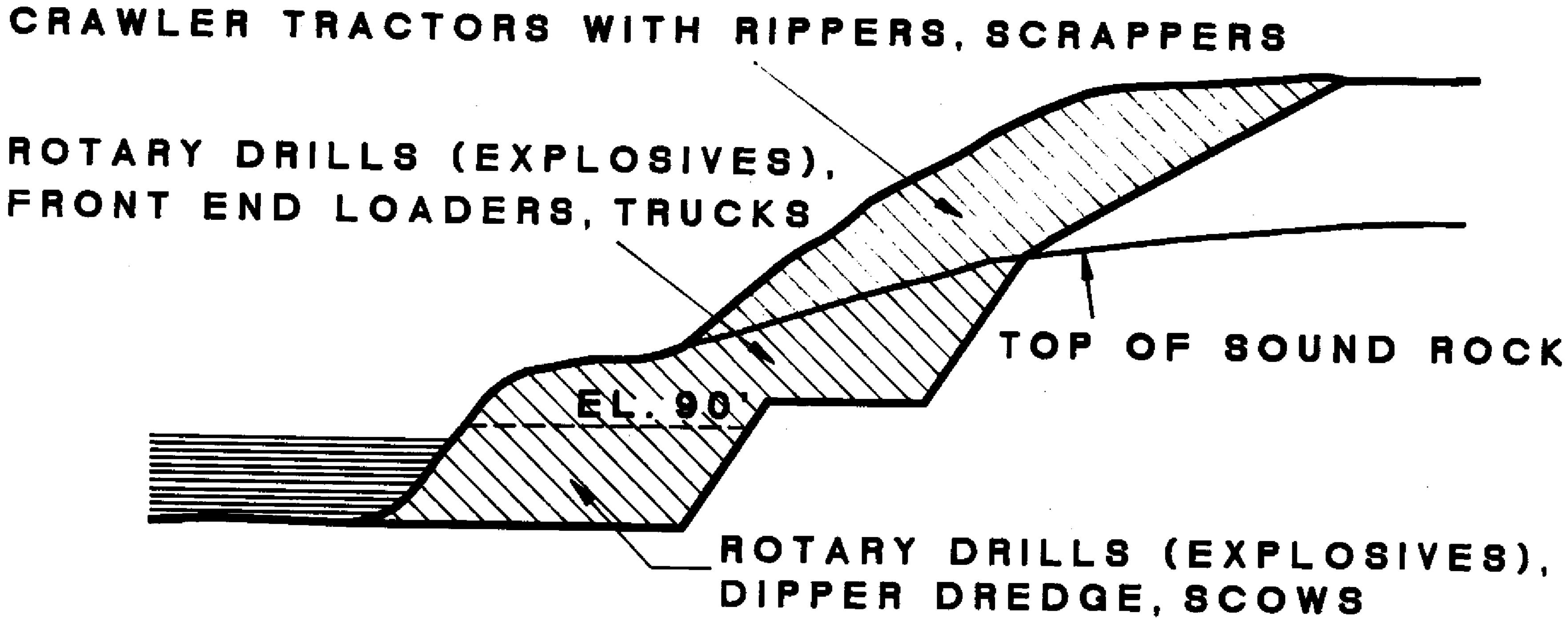
Construction will begin at the north end of the project. After disposal-area preparation and construction of access and haul roads for the northernmost areas, "dry" excavation will commence and progress southward. When this work has sufficiently advanced, "wet" excavation will commence and follow southward (Figure 12). The dimensions of the two dipper dredges and the large hopper barges, which would be used to transport the excavated material to the open water disposal areas, preclude operating them alongside each other within the 130' access corridor that constitutes the minimum widening area. Therefore, it will be necessary for one of the two dipper dredges to excavate a portion of the widening strip and proceed southward for a reasonable distance. At this point, the second dipper barge could remove the remaining strip for the access corridor and th material which sloughs into the widened section from the slopes. This sequence would allow the productive usage of both dipper dredges without creating any interference to the traffic movement through the existing canal. "Dry" work can be carried on concurrently in other areas.

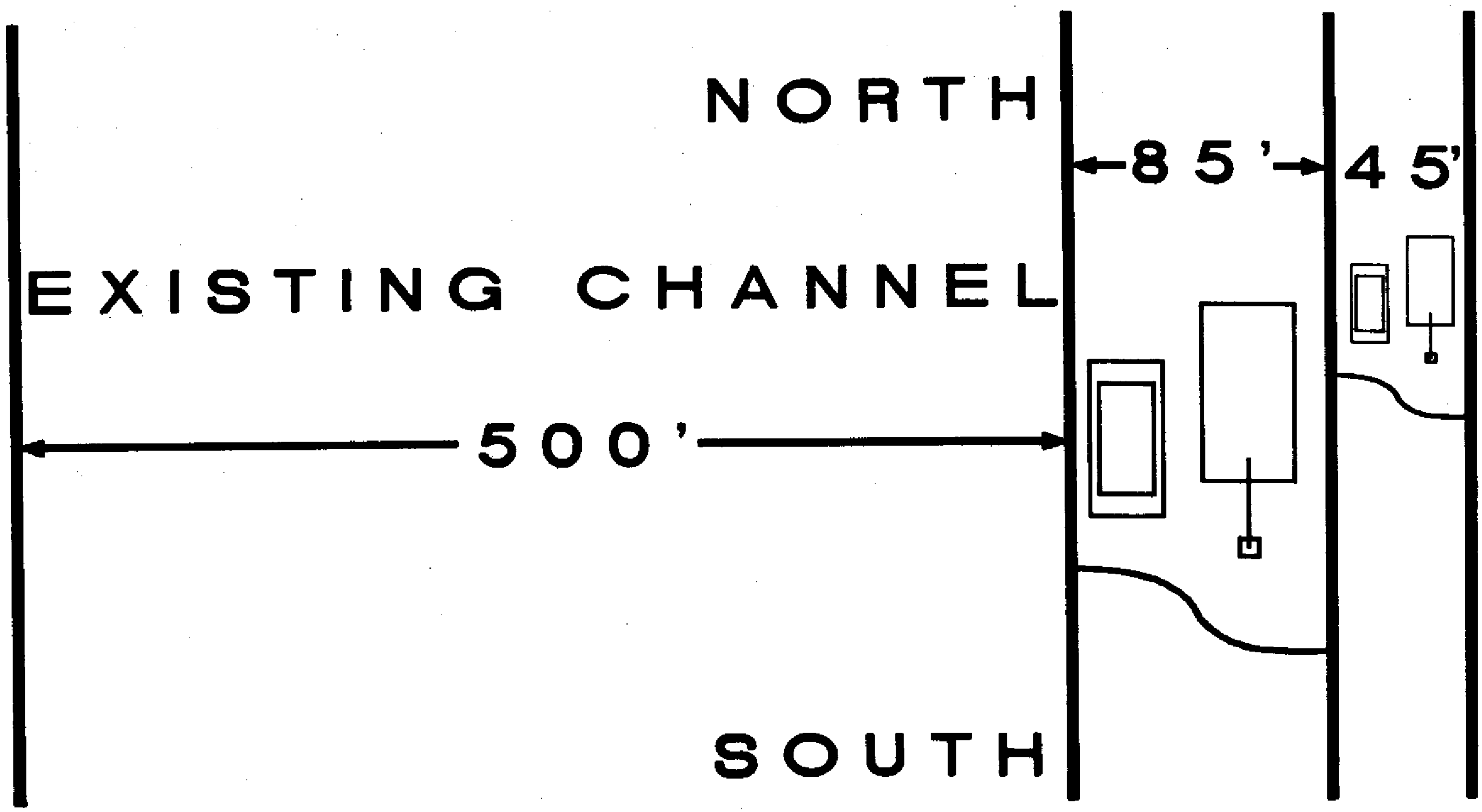
Cost — Cost estimates included in Figure 13 are in 1986 dollars and pertain to one prime contractor with a 5-year construction period.

Cost
Figure 13

	Million $
Preparation of Area	8
Drilling and Blasting	33
Loading and Hauling	62
Loading and Towing	90
Gold Hill Special Consideration	10
General Condition	48
Mobilization and Demobilization	7
Profit	26
Contingencies (20%)	57
Commission Support	59
Total	400

EXCAVATION METHODOLOGY





LARGE DIPPER DREDGE

SMALL DIPPER DREDGE

Preparation of the area includes cleaning, grubbing, road construction and maintenance, and ripping.

Most of the 433 million estimated from drilling and blasting is for wet materials: land-based accounting for $20 million and marine-based for $3 million. The other $10 million is for dry material.

The loading and hauling estimate assumes that one-third of the approximated 8.5 million cubic yards of dry earth and weathered rock excavations will be performed with scrapers. Fourteen million cubic yards of dry sound rock and approximately 6 million cubic yards of the dry earth and weathered rock will be handled by loader/trucks.

Dipper dredges will load bottom-dump scows. These will be towed to Gatun Lake disposal area and dumped. Towing distances vary from 13 to 20 miles.

In order to accomplish the excavation at Gold Hill, an estimated additional cost of $5.00 per cubic yard is included for the approximately 2 million cubic yards of "dry" excavation required. This represents the equivalent of additional cost for this work, over and above that already included in other line items covering the total volume.

The cost for general conditions is based on the assumption that the large contracting agency will perform the dry potion with its own forces, subcontracting the subaqueous work to a firm with dipper-dredge capability. One-third of the total is for home office expense.

It is assumed that land-based equipment will be hauled or driven from the home yard to a United States port, shipped to Panama by ocean freight, unloaded at the Panama port and hauled or driven to the work site. Upon project completion, equipment will be demobilized to the United States in a manner similar to mobilization.

A profit rate is estimated using the Corps of Engineers "weighted guidelines" method, which covers most of the significant variable factors which should be considered such as risk, difficulty, contractor's investment, and job size.

Contingencies have been estimated at 20%, while commission support (completion of design, contract administration and relocation of facilities) has been estimated at 15 to 20 percent.

The plan just presented is considered to be most cost-effective of all those considered. A single contract and a continuing-construction schedule will be most efficient from a performance standpoint, because the planning, coordination and scheduling of operations for the entire project would be the responsibility of a single contractor. Also, the large, special-purpose equipment necessary for the project would have to be mobilized only once and could be moved about to meet project needs at any given time during the construction period.

Cost of Alternative Schedules — Figure 14 shows information on construction schedules which wold exceed the optimum construction plan and schedule. This information is being used by the commissions to assess the effects of variations in the financial requirements of the project.

Project Cost for Major Schemes
Figure 14

Description of Scheme	Cost (Millions)
1. One Contractor: 5 years	400
2. One Contractor per sector:	
7 years	430
11 years	455
3. One Contractor for Dry and One For Wet Excavation by Sector:	
7 years	485
11 years	515
4. Wet excavation by PCC and One Contractor for Dry by Sector:	
11 years	410

The use of three separate and sequential contracts to achieve the work progressively from the northern section through the central sector, and lastly the southern section would increase the optimum-plan cost and construction period. The construction-period increase is due to the fact that the dredging activity must be completed before the next phase can get fully underway.

The option of extending the contract period wold also result in an increase in the optimum-plan cost. This cost increase is due to equipment maintenance and support personnel for the additional period of time. The use of smaller, less specialized equipment over the longer period has no significant effect on project costs.

Environmental Analysis

Basis for Evaluation — Evaluation of potential ecological impacts is a major part of assessing the overall feasibility of any waterway improvement project. Our environmental review reflects both federal guidance and treaty-derived commitments. We have worked closely on this report with corps of engineers experts and the environment organizations in Panama.

Areas of Concern — The potential environmental impacts of the project are mainly determined by the types of equipment needed, by the amounts and nature of rock or sol excavated and disposed, and by the construction periods, whether work is performed continuously or intermittently.

All relevant environmental parameters were studied, as listed in the second part Figure 15.

Primary Areas of Concern
Figure 15

Elements Affecting Environment
Excavation and Disposal Methods
Volumes and Characteristics of Material
Time Frames

Parameters Studied
Vegetation and Wildlife
Air, Water, and Noise
Cultural and Archaeological Resources
Social and Economic Conditions

Upland Disposal Selection (Figure 16) — Under the recommended project plan, virtually all the material from the excavation work performed on land would be disposed on-land.

Selection of the appropriate upland disposal sites was an important part of our environmental review.

Upland Disposal Site Selection
Figure 16

Engineering Criteria
Haul Distance and Accessibility
Site Capacity
Site Preparation and Drainage

Environmental Characteristics
Presence of Plants and Animals
Current, Past and Projected Uses
Proximity to Protected Areas and Townsites
Cultural and Archaeological

Underwater Disposal Selection (Figure 17) — The marine-based excavations (or dredging-type operations) associated with widening require disposal in open-water dumps. This type of disposal constitutes approximately one-third of the total material that would be excavated for the project. The same site selection criteria were followed as for the upland areas. The main engineering concerns included haul distance. Because of its cost constraints, and, again, assurances that sufficient capacity would be available. Environmental concerns centered on the biological and chemical characteristics of the water and bottom sediment.

Underwater Disposal Site Selection
Figure 17

Engineering Criteria
Haul Distance
Capacity (Deepest Areas Preferred)

Environmental Characteristics
Water Quality
Sediment Composition
Aquatic Life

Conclusions — In summary, the widening operations do not introduce any significant negative elements into the canal environment, in terms of equipment, procedures, areas affected, and other potential sources of ecological concern. We have concluded, based on the review, that the proposed action would not result in an adverse impact within the meaning of the National Environmental Policy Act.

Economic Analysis

Objective — The objective of the economic analysis is the assessment of the economic feasibility of the project.

Considerations — Revenues generated from traffic in the form of toll revenues, transit-related revenues and intangibles are the inflows of funds while investment and expenses represent the outlays of cash.

Additional revenues and benefits from widening are generated by increasing capacity and tolls and the cost avoidance of reducing ship's time spent in canal waters.

For the project to be feasible, the revenues and benefits should more than compensate for the cost of the investment and for the additional operating expenses resulting from the project.

Final results of the economic analysis are pending the completion of the financial analysis where the nature, sources and cost of funds required for the project will be determined.

Financial Analysis

Objective — In the financial analysis several funding alternatives are considered and the best one selected from among the feasible options.

Investment strategies — The several financial strategies spelling construction periods resulted from the interaction of traffic forecast, the identification of the capacity pinchpoint identified in the operational analysis, and finally the construction periods and cost identified by the Corps of Engineers.

Figure 18 includes not only the forecast projected by Manalytics, but also the contingency that the forecast may be off by two daylight vessels. The nine options offered cover the wide range of possibilities available. As a way of example, for a five-year construction period and arrivals behaving as predicted by Manalytics, the project should start later than 1995 and be completed no later than the year 2000.

Investment Strategies
Figure 18

Forecast	Construction Period (Yrs) 5 LS	LC	7 LS	LC	11 LS	LC
Manalytics +2DL	1991	1996	1990	1997	1989	2000
Manalytics	1995	2000	1995	2002	1994	2005
Manalytics	2001	2006	2000	2007	1999	2010

Note: LS = Latest Start
LC = Latests Completion

Elements — Financial alternatives are based on the proportion of equity obtained from tolls and the loan required for the project. Tolls could be raised to the optimum level determined by the toll sensitivity analysis, and the loan requirement determined as a dropout figure from the total monetary requirement less the amount financed from tolls.

Inasmuch as Panama will take over the ownership and operation of the canal in the year 2000, the duration of the loan is a national concern.

The level at which the toll rate is set must take into consideration the sensitivity of the users as well as the impact on the Panama Canal Commission, the Republic of Panama and the United States.

Study Completion

Upon completion of this study targeted for 1987, a recommendation will be presented to the Board of Directors. Because of the vested interest of United States and Panama in this project, it is likely that its final disposition will be discussed and agreed upon at diplomatic levels.

The Texas A&M University Sea Grant College Program publishes technical reports, proceedings and curricular supplements that contribute to its mandate of furthering research, education and public awareness of the oceans and the marine environment. All manuscripts are peer reviewed by appropriate experts in the respective fields, and made available to the general public at cost. Inquiries regarding potential publications should be directed to the Sea Grant Program Director or the Marine Information Service Editor.